<table>
<tr><td>Course Code:</td><td rowspan="2">HYDROLOGY & WATER RESOURCES ENGINEERING</td><td>L</td><td>T</td><td>P</td><td>Credit</td><td>CIA Marks 40</td></tr>
<tr><td>Course Category:
PCC</td><td>2</td><td>0</td><td>2</td><td>3</td><td>SEE Marks 60</td></tr>
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COURSE OBJECTIVES:

- To study occurrence movement and distribution of water that is a prime resource for development of a civilization..
- To know diverse methods of collecting the hydrological information, which is essential, to understand surface and ground water hydrology.
- To know the basic principles and movement of ground water and properties of groundwater flow.

COURSE OUTCOMES:

- Provide a background in the theory of hydrological processes and their measurement
- Apply science and engineering fundamentals to solve current problems and to anticipate, mitigate and prevent future problems in the area of water resources management
- An ability to manipulate hydrological data and undertake widely-used data analysis.
- Can define the key components of a functioning groundwater, can determine the main aquifer properties – permeability, transmissivity and storage Identify geological formations capable of storing and transporting groundwater.
- Different methods and importance of rain water harvesting.

After learning the course the students should be able to:

COS NO.	**Course Outcomes**	**Bloom's level**
CO1	Provide a background in the theory of hydrological processes and their measurement	Understand
CO2	Apply science and engineering fundamentals to solve current problems and to anticipate, mitigate and prevent future problems in the area of water resources management	Apply
CO3	An ability to manipulate hydrological data and undertake widely-used data analysis.	Analysis
CO4	Can define the key components of a functioning groundwater, can determine the main aquifer properties – permeability, transmissivity and storage Identify geological formations capable of storing and transporting groundwater.	Understand, Analysis
CO5	Different methods and importance of rain water harvesting	Apply

Mapping of Course Outcome to Program Outcomes:

Mapping of COs with POs												
COs/POs	PO1	PO2	PO3	PO4	PO5	PO6	PO7	PO8	PO9	PO10	PO11	PO12
CO1	L	L	L	L	L	S	M	L	L	M	S	S
CO2	M	M	M	L	L	M	M	M	L	M	S	M
CO3	L	L	L	M	S	L	L	L	M	M	S	S
CO4	L	S	---	L	---	S	S	M	L	M	S	M
CO5	L	M	L	L	M	M	M	L	L	L	M	M

SYLLABUS:

MODULE I

INTRODUCTION

Hydrologic cycle, Climate and water availability, Water balances, Precipitation: Forms, Classification, Variability, Measurement, Data analysis, Evaporation and its measurement, Evapo-transpiration and its measurement, Penman Monteith method. Infiltration: Factors affecting infiltration, Horton's equation and Green Ampt method.

MODULE II

HYETOGRAPH AND HYDROGRAPH ANALYSIS

Hyetograph, Runoff: drainage basin characteristics, Hydrograph concepts, assumptions and limitations of unit hydrograph, Derivation of unit hydrograph, S- hydrograph, Flow duration curve

Groundwater: Occurrence, Darcy's law, Well hydraulics, Well losses, Yield, Pumping and recuperation test

MODULE III

RESERVOIR AND HYDROELECTRIC POWER

Reservoir: Types, Investigations, Site selection, Zones of storage, Safe yield, Reservoir capacity, Reservoir sedimentation and control. Introduction to Dams Introduction and types of dams, spillways and ancillary works, Site assessment and selection of type of dam, Information about major dams and reservoirs of India

Hydroelectric Power: Low, Medium and High head plants, Power house components, Hydel schemes

MODULE IV

FLOOD MANAGEMENT and HYDROLOGIC ANALYSIS

Flood Management: Indian rivers and floods, Causes of floods, Alleviation, Leeves and floodwalls, Floodways, Channel improvement, Flood damage analysis.

Hydrologic Analysis: Design flood, Flood estimation, Frequency analysis, Flood routing through reservoirs and open channels.

MODULE V

DROUGHT MANAGEMENT AND WATER HARVESTING

Definition of drought, Causes of drought, measures for water conservation and augmentation, drought contingency planning. Water harvesting: rainwater collection, small dams, runoff enhancement, runoff collection, ponds, tanks.

LEARNING RESOURCES:

Text Books

1. K Subramanya, Engineering Hydrology, Mc-Graw Hill. New Delhi.

2. K N Muthreja, Applied Hydrology, Tata Mc-Graw Hill.

3. K Subramanya, Water Resources Engineering through Objective Questions, Tata Mc-

Graw Hill.

4. G L Asawa, Irrigation Engineering, Wiley Eastern

References:

1. L W Mays, Water Resources Engineering, Wiley.
2. J D Zimmerman, Irrigation, John Wiley & Sons
3. C S P Ojha, R Berndtsson and P Bhunya, Engineering Hydrology, Oxford.
4. R.K. Sharma and T.K. Sharma, Hydrology and Water Resources Engineering, Prentice Hall of India, New Delhi.

Introduction of Hydrology

HYDROLOGY:-

Hydrology is the science, which deals with the occurrence, distribution and disposal of water on the planet earth.

Hydro=Water Logy=Science

HYDROLOGIC CYCLE

Hydrologic cycle is the water transfer cycle, which occurs continuously in nature; the three important phases of the hydrologic cycle are: (a) Evaporation and evapotranspiration (b)precipitation and (c) runoff

Evaporation from the surfaces of ponds, lakes, reservoirs. ocean surfaces, etc. and transpiration from surface vegetation i.e., from plant leaves of cropped land and forests, etc. take place. These vapours rise to the sky and are condensed at higher altitudes by condensation nuclei and form clouds, resulting in droplet growth. The clouds melt and sometimes burst resulting in precipitation of different forms like rain, snow, hail, sleet, mist, dew and frost. A part of this precipitation flows over the land called runoff and part infilters into the soil which builds up the ground water table. The surface runoff joins the streams and the water is stored in reservoirs. A portion of surface runoff and ground water flows back to ocean. Again evaporation starts from the surfaces of lakes, reservoirs and ocean, and the cycle repeats. Of these three phases of the hydrologic cycle, namely, evaporation, precipitation and runoff, it is the 'runoff phase', which is important to a civil engineer since he is concerned with the storage of surface runoff in tanks and reservoirs for the purposes of irrigation, municipal water supply hydroelectric power etc.

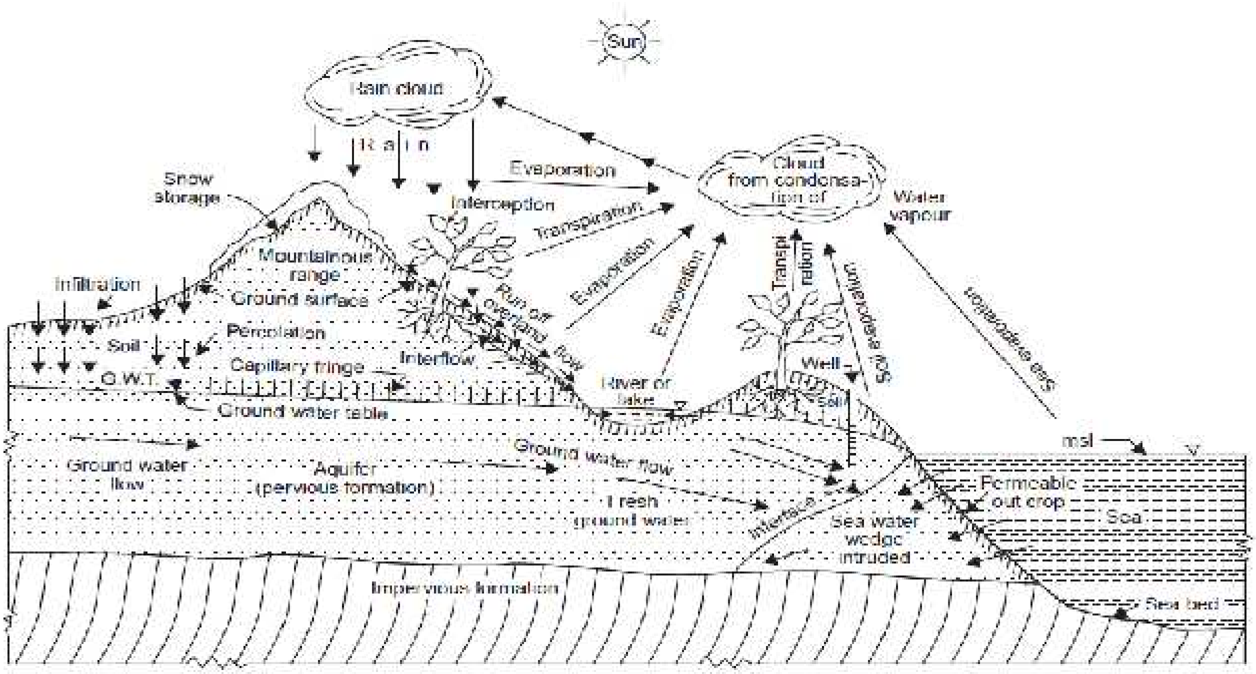


Fig. 1.7 The hydrologic cycle

FORMS OF PRECIPITATION

Drizzle — a light steady rain in fine drops (0.5 mm) and intensity <1 mm/hr

Rain — the condensed water vapour of the atmosphere falling in drops (>0.5 mm, maximum size—6 mm) from the clouds.

Glaze — freezing of drizzle or rain when they come in contact with cold objects.

Sleet — frozen rain drops while falling through air at subfreezing temperature.

Snow — ice crystals resulting from sublimation (i.e., water vapour condenses to ice)

Snow flakes — ice crystals fused together.

Hail — small lumps of ice (>5 mm in diameter) formed by alternate freezing and melting, when they are carried up and down in highly turbulent air currents.

Dew — moisture condensed from the atmosphere in small drops upon cool surfaces.

Frost — a feathery deposit of ice formed on the ground or on the surface of exposed objects by dew or water vapour that has frozen

Fog — a thin cloud of varying size formed at the surface of the earth by condensation of atmospheric vapour (interfering with visibility)

Mist — a very thin fog

SCOPE OF HYDROLOGY

The study of hydrology helps us to know

(i)the maximum probable flood that may occur at a given site and its frequency; this is required for the safe design of drains and culverts, dams and reservoirs, channels and other flood control structures.

(ii) the water yield from a basin—its occurrence, quantity and frequency, etc; this is necessary for the design of dams, municipal water supply, water power, river navigation, etc.

(iii) the ground water development for which a knowledge of the hydrogeology of the area, i.e., of the formation soil, recharge facilities like streams and reservoirs, rainfall pattern, climate, cropping pattern, etc. are required.

(iv) the maximum intensity of storm and its frequency for the design of a drainage project in the area.

TYPES OF PRECIPITATION

The precipitation may be due to

(i) ***Thermal convection (convectional precipitation)***
This type of precipitation is in the form of local whirling thunder storms and is typical of the tropics. The air close to the warm earth gets heated and rises due to its low density, cools adiabatically to form a cauliflower shaped cloud, which finally bursts into a thunder storm. When accompanied by destructive winds, they are called 'tornados'.

(ii) ***Conflict between two air masses (frontal precipitation)***
When two air masses due to contrasting temperatures and densities clash with each other, condensation and precipitation occur at the surface of contact, Fig. 2.1. This surface of contact is called a 'front' or 'frontal surface'. If a cold air mass drives out a warm air mass' it is called a 'cold front' and if a warm air mass replaces the retreating cold air mass, it is called a 'warm front'. On the other hand, if the two air masses are drawn simultaneously towards a low pressure area, the front developed is stationary and is called a 'stationary front'. Cold front causes intense precipitation on comparatively small areas, while the precipitation due to warm front is less intense but is spread over a comparatively larger area. Cold fronts move faster than warm fronts and usually overtake them, the frontal surfaces of cold and warm air sliding against each other. This phenomenon is called 'occlusion' and the resulting frontal surface is called an 'occluded front'.

(iii) ***Orographic lifting (orographic precipitation****)*
The mechanical lifting of moist air over mountain barriers, causes heavy precipitation on the windward side (Fig. 2.2). For example Cherrapunji in the Himalayan range and Agumbe in the western Ghats of south India get very heavy orographic precipitation of 1250 cm and 900 cm (average annual rainfall), respectively.

(iv) ***Cyclonic (cyclonic precipitation****)*
This type of precipitation is due to lifting of moist air converging into a low pressure belt, i.e., due to pressure differences created by the unequal heating of the earth's surface. Here the winds blow spirally inward counterclockwise in the northern hemisphere and clockwise in the southern hemisphere. There are two main types of cyclones—tropical cyclone (also called hurricane or typhoon) of comparatively small diameter of 300-1500 km causing high wind velocity and heavy precipitation, and the extra-tropical cyclone of large diameter up to 3000 km causing wide spread frontal type precipitation.

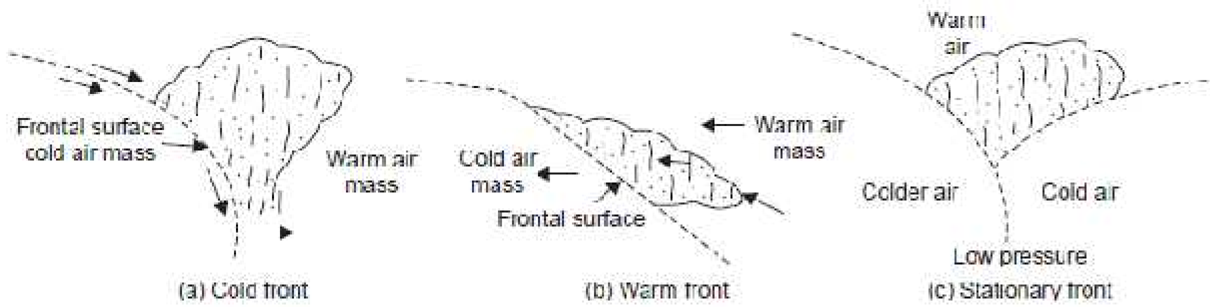


Fig. 2.1 Frontal precipitation

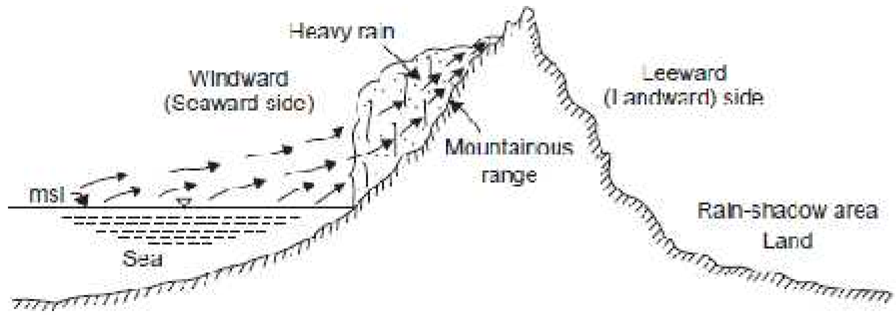
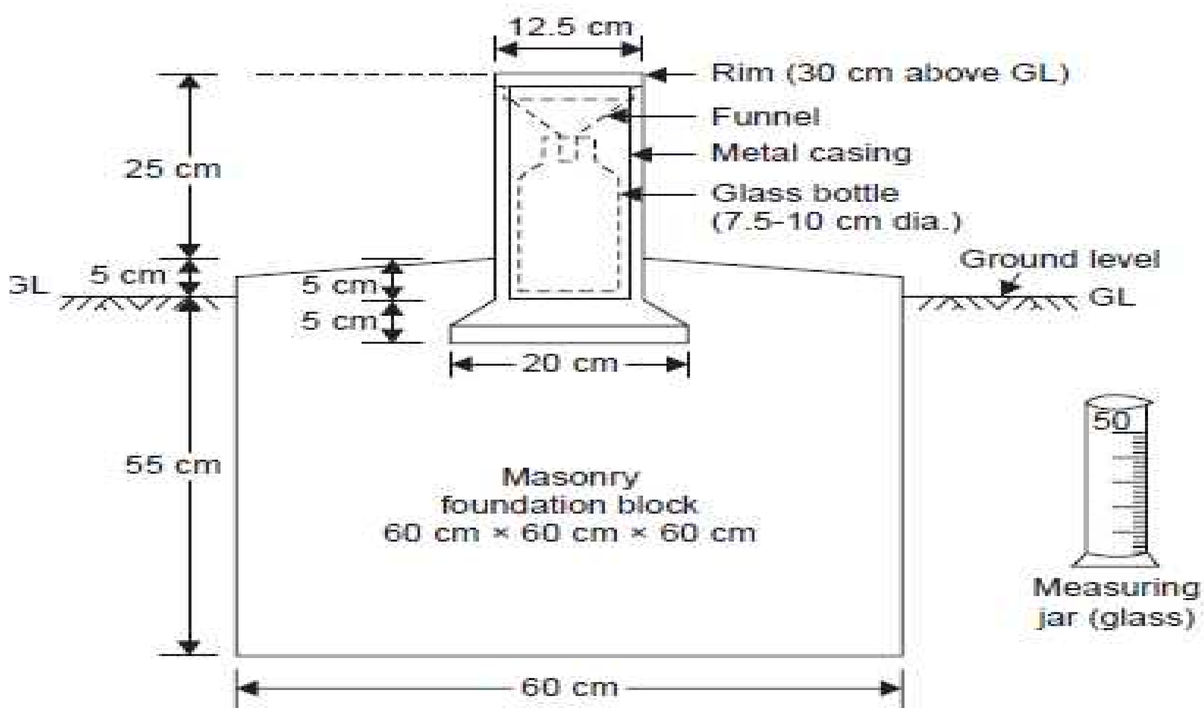


Fig. 2.2 Orographic precipitation

MEASUREMENT OF PRECIPITATION

Rainfall may be measured by a network of rain gauges which may either be of non-recording or recording type.



The non-recording rain gauge used in India is the Symon's rain gauge (Fig. 2.3). It consists of a funnel with a circular rim of 12.7 cm diameter and a glass bottle as a receiver. The cylindrical metal casing is fixed vertically to the masonry foundation with the level rim 30.5 cm above the ground surface. The rain falling into the funnel is collected in the receiver and is measured in a special measuring glass graduated in mm of rainfall; when full it can measure 1.25 cm of rain. The rainfall is measured every day at 08.30 hours IST. During heavy rains, it must be measured three or four times in the day, lest the receiver fill and overflow, but the last measurement should be at 08.30 hours IST and the sum total of all the measurements during the previous 24 hours entered as the rainfall of the day in the register. Usually, rainfall measurements are made at 08.30 hr IST and sometimes at 17.30 hr IST also. Thus the non-recording or the Symon's rain gauge gives only the total depth of rainfall for the previous 24 hours (i.e., daily rainfall) and does not give the intensity and duration of rainfall during different time intervals of the day. It is often desirable to protect the gauge from being damaged by cattle and for this purpose a barbed wire fence may be erected around it.

Recording Rain Gauge

This is also called self-recording, automatic or integrating rain gauge. This type of rain gauge Figs. 2.4, 2.5 and 2.6, has an automatic mechanical arrangement consisting of a clockwork, a drum with a graph paper fixed around it and a pencil point, which draws the mass curve of rainfall Fig. 2.7. From this mass curve, the depth of rainfall in a given time, the rate or intensity of rainfall at any instant during a storm, time of onset and cessation of rainfall, can be determined. The gauge is installed on a concrete or masonry platform 45 cm square in the observatory enclosure by the side of the ordinary rain gauge at a distance of 2-3 m from it. The gauge is so installed that the rim of the funnel is horizontal and at a height of exactly 75 cm above ground surface. The self-recording rain gauge is generally used in conjunction with an ordinary rain gauge exposed close by, for use as standard, by means of which the readings of the recording rain gauge can be checked and if necessary adjusted.

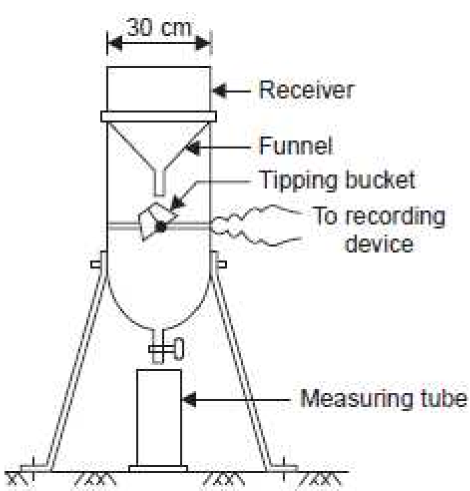


Fig. 2.4 Tipping bucket gauge

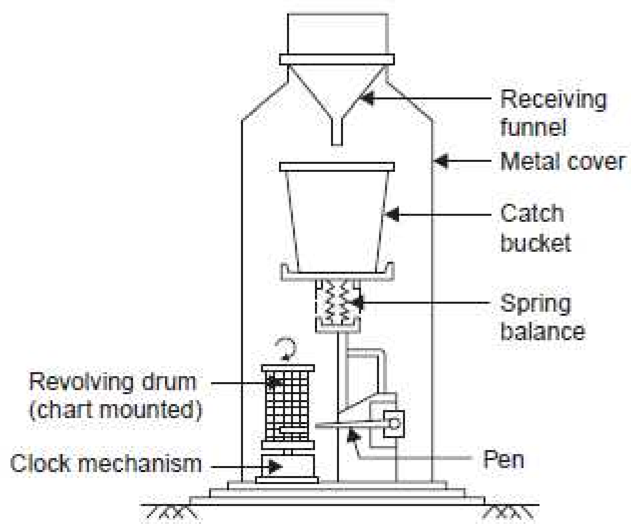


Fig. 2.5 Weighing type rain gauge

There are three types of recording rain gauges—tipping bucket gauge, weighing gauge and float gauge.

Tipping bucket rain gauge.

This consists of a cylindrical receiver 30 cm diameterwith a funnel inside (Fig. 2.4). Just below the funnel a pair of tipping buckets is pivoted such that when one of the bucket receives a rainfall of 0.25 mm it tips and empties into a tankbelow, while the other bucket takes its position and the process is repeated. The tipping of the bucket actuates on electric circuit which causes a pen to move on a chart wrapped round a drum which revolves by a clock mechanism. This type cannot record snow.

Weighing type rain gauge. *In this type of rain-gauge, when a certain weight of rainfall is collected in a tank, which rests on a spring-lever balance, it makes a pen to move on a chart wrapped round a clockdriven drum (Fig. 2.5). The rotation of the drum sets the time scale while the vertical motion of the pen records the cumulative precipitation.*

Float type rain gauge. *In this type, as the rain is collected in a float chamber, the float moves up which makes a pen to move on a chart wrapped round a clock driven drum (Fig. 2.6). When the float chamber fills up, the water siphons out automatically through a siphon tube kept in an interconnected siphon chamber. The clockwork revolves the drum once in 24 hours. The clock mechanism needs rewinding once in a week when the chart wrapped round the drum is also replaced. This type of gauge is used by IMD.*

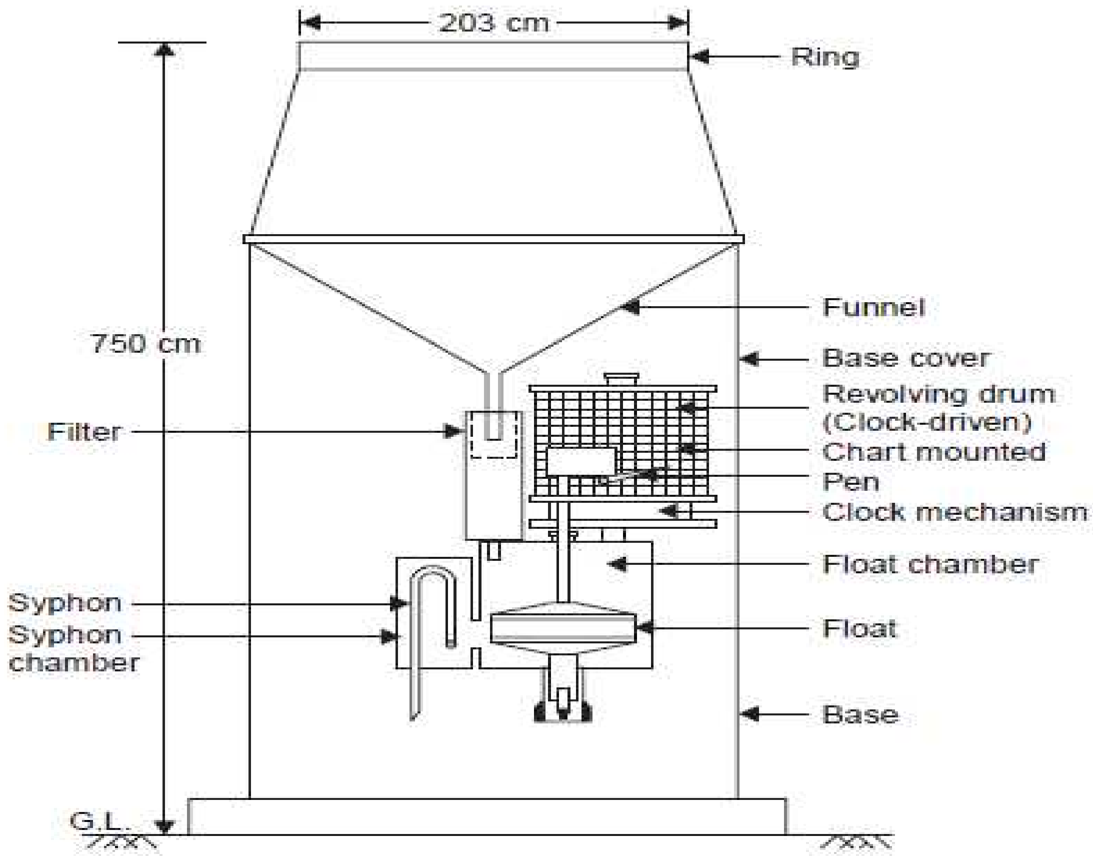


Fig. 2.6 Float type rain gauge

The weighing and float type rain gauges can store a moderate snow fall which the operator can weigh or melt and record the equivalent depth of rain.

ESTIMATE OF MISSING RAINFALL DATA

(i) ***Station-year method***

In this method, the records of two or more stations are combined into one long record provided station records are independent and the areas in which the stations are located are climatologically the same. The missing record at a station in a particular year may be found by the ratio of averages or by graphical comparison. For example, in a certain year the total rainfall of station A is 75 cm and for the neighbouring station B, there is no record. But if the a.a.r. at A and B are 70 cm and 80 cm, respectively, the missing year's rainfall at B (say, PB) can be found by simple proportion as:

$$PB = 85.7 \text{ cm}$$

This result may again be checked with reference to another neighbouring station C.

(ii) **By simple proportion (normal ratio method)**

.

(iii) ***Double-mass analysis***

The trend of the rainfall records at a station may slightly change after some years due to a change in the environment (or exposure) of a station either due to coming of a new building, fence, planting of trees or cutting of forest nearby, which affect the catch of the gauge due to change in the wind pattern or exposure. The consistency of records at the station in question (say, X) is tested by a double mass curve by plottting the cumulative annual (or seasonal) rainfall at station X against the concurrent cumulative values of mean annual (or seasonal) rainfall for a group of surrounding stations, for the number of years of record (Fig. 2.9). From the plot, the year in which a change in regime (or environment) has occurred is indicated by the change in slope of the straight line plot. The rainfall records of the station x are adjusted by multiplying the recorded values of rainfall by the ratio of slopes of the straight lines before and after change in environment.

WATER LOSSES:

The hydrologic equation states that

Rainfall–Losses = Runoff

(i) Interception loss-due to surface vegetation, i.e., held by plant leaves.
(ii) Evaporation:
(a) from water surface, i.e., reservoirs, lakes, ponds, river channels, etc.
(b) from soil surface, appreciably when the ground water table is very near the soil surface.
(iii) Transpiration—from plant leaves.
(iv) Evapotranspiration for consumptive use—from irrigated or cropped land.
(v) Infiltration—into the soil at the ground surface.
(vi) Watershed leakage—ground water movement from one basin to another or into the sea.
The various water losses are discussed below:

Interception loss—

The precipitation intercepted by foliage (plant leaves, forests) and buildings and returned to atmosphere (by evaporation from plant leaves) without reaching the ground surface is called interception loss. Interception loss is high in the beginning of storms and gradually decreases; the loss is of the order of 0.5 to 2 mm per shower and it is greater in the case of light showers than when rain is continuous. Fig. 3.1 shows the Horton's mean curve of interception loss for different showers.

Effective rain = Rainfall – Interception loss

EVAPORATION:

Evaporation from free water surfaces and soil are of great importance in hydro-meterological studies.

Evaporation from water surfaces (Lake evaporation):

The factors affecting evaporation are air and water temperature, relative humidity, wind velocity, surface area (exposed), barometric pressure and salinity of the water, the last two having a minor effect. The rate of evaporation is a function of the differences in vapour pressure at the water surface and in the atmosphere, and the Dalton's law of evaporation is given by

$$E = K\,(e_w - e_a)$$

where E = daily evaporation
ew = saturated vapour pressure at the temperature of water
ea = vapour pressure of the air (about 2 m above)
K = a constant.

The Dalton's law states that the evaporation is proportional to the difference in vapour pressures e_w and e_a. A more general form of the Eq. (3.2) is given by

$$E = K'\,(e_w - e_a)\,(a + bV)$$

where K', a, b = constants
and V = wind velocity.

EVAPORATION PANS:

***(i) Floating pans** (made of GI) of 90 cm square and 45 cm deep are mounted on a raft floating in water. The volume of water lost due to evaporation in the pan is determined by knowing the volume of water required to bring the level of water up to the original mark daily and after making allowance for rainfall, if there has been any.*

***(ii) Land pan.** Evaporation pans are installed in the vicinity of the reservoir or lake to determine the lake evaporation. The IMD Land pan shown in Fig. 3.2 is 122 cm diameter and 25.5 cm deep, made of unpainted GI; and set on wood grillage 10 cm above ground to permit circulation of air under the pan. The pan has a stilling well, vernier point gauge, a thermometer with clip and may be covered with a wire screen. The amount of water lost by evaporation from the pan can be directly measured by the point gauge. Readings are taken twice daily at 08.30 and 17.30 hours I.S.T. The air temperature is determined by reading a dry bulb thermometer kept in the Stevenson's screen erected in the same enclosure of the pan. A totalising anemometer is normally mounted at the level of the instrument to provide the wind speed information*

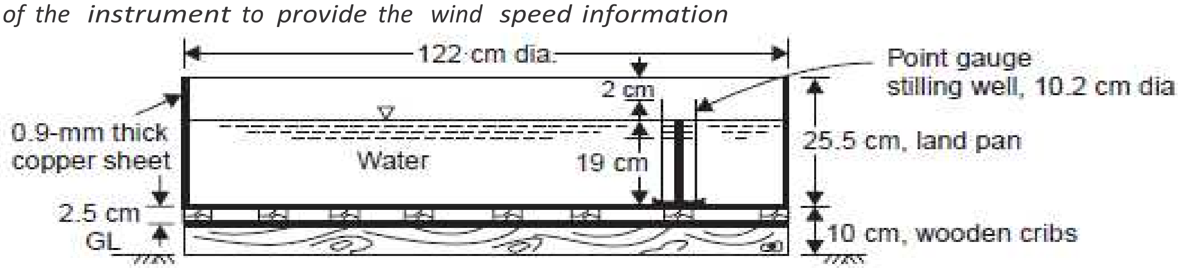


Fig. 3.2 IMD land pan

required. Allowance has to be made for rainfall, if there has been any. Water is added to the pan from a graduated cylinder to bring the water level to the original mark, i.e., 5 cm below the top of the pan. Experiments have shown that the unscreened pan evaporation is 1.144 times that of the screened one.

(iii) Colarado sunken pan. *This is 92 cm square and 42-92 cm deep and is sunk in the ground such that only 5-15 cm depth projects above the ground surface and thus the water level is maintained almost at the ground level. The evaporation is measured by a point gauge*

Pan coefficient—

Evaporation pan data cannot be applied to free water surfaces di- rectly but must be adjusted for the differences in physical and climatological factors. For ex- ample, a lake is larger and deeper and may be exposed to different wind speed, as compared to a pan. The small volume of water in the metallic pan is greatly affected by temperature fluc- tuations in the air or by solar raditions in contrast with large bodies of water (in the reservoir) with little temperature fluctuations. Thus the pan evaporation data have to be corrected to obtain the actual evaporation from water surfaces of lakes and reservoirs, i.e., by multiplying by a coefficient called pan coefficient

TRANSPIRATION:

Tanspiration is the process by which the water vapour escapes from the living plant leaves and enters the atmosphere. Various methods are devised by botanists for the measurement of transpiration and one of the widely used methods is by phytometer. It consists of a closed water tight tank with sufficient soil for plant growth with only the plant exposed; water is applied artificially till the plant growth is complete. The equipment is weighed in the begin- ning (W_1) and at the end of the experiment (W_2). Water applied during the growth (w) is measured and the water consumed by transpiration (W_t) is obtained as

$$W_t = (W_1 + w) - W_2$$

The experimental values (from the protected growth of the plant in the laboratory) have to be multiplied by a coefficient to obtain the possible field results.

Transpiration ratio is the ratio of the weight of water absorbed (through the root system), conveyed through and transpired from a plant during the growing season to the weight of the dry matter produced exclusive of roots.

For the weight of dry matter produced, sometimes, the useful crop such as grains of wheat, gram, etc. are weighed. The values of transpiration ratio for different crops vary from 300 to 800 and for rice it varies from 600 to 800 the average being 700.

Evaporation losses are high in arid regions where water is impounded while transpira- tion is the major water loss in humid regions.

EVAPOTRANSPIRATION

Evapotranspiration (E_t) or consumptive use (U) is the total water lost from a cropped (or irrigated) land due to evaporation from the soil and transpiration by the plants or used by the plants in building up of plant tissue. Potential evapotranspiration (E_{pt}) is the evapotranspiration from the short green vegetation when the roots are supplied with unlimited water covering the soil. It is usually expressed as a depth (cm, mm) over the area.

Estimation of Evapotranspiration

The following are some of the methods of estimating evapotranspiration:

(i) Tanks and lysimeter experiments

(ii) Field experimental plots

(iii) Installation of sunken (colarado) tanks

(iv) Evapotranspiration equations as developed by Lowry-Johnson, Penman, Thornthwaite, Blaney-Criddle, etc.

(v) Evaporation index method, i.e., from pan evaporation data as developed by Hargreaves and Christiansen.

Factors Affecting Evapotranspiration

From the above equations it can be seen that the following factors affect the evapotranspiration:

(i) Climatological factors like percentage sunshine hours, wind speed, mean monthly temperature and humidity.

(ii) Crop factors like the type of crop and the percentage growing season.

(iii) The moisture level in the soil.

INFILTRATION:

Water entering the soil at the ground surface is called infiltration. It replenishes the soil mois- ture deficiency and the excess moves downward by the force of gravity called deep seepage or percolation and builds up the ground water table. The maximum rate at which the soil in any given condition is capable of absorbing water is called its infiltration capacity (f_p). Infiltration (f) often begins at a high rate (20 to 25 cm/hr) and decreases to a fairly steady state rate (f_c) as the rain continues, called the ultimate f_p (= 1.25 to 2.0 cm/hr) (Fig. 3.6). The infiltration rate (f) at any time t is given by Horton's equation.

$$f = f_c + (f_o - f_c)\, e^{-kt}$$

$$k = \frac{f_0 - f_c}{F_c}$$

where f_0 = initial rate of infiltration capacity

f_c = final constant rate of infiltration at saturation

k = a constant depending primarily upon soil and vegetation

e = base of the Napierian logarithm

F_c = shaded area in Fig. 3.6

t = time from beginning of the storm

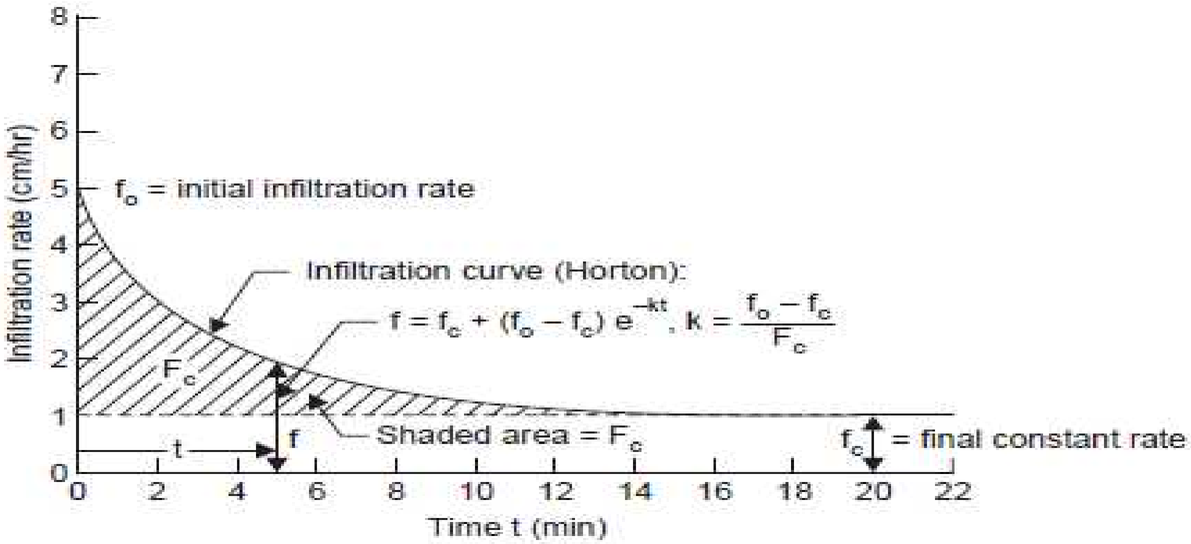


Fig. 3.6 Infiltration Curve (Horton)

The infiltration takes place at capacity rates only when the intensity of rainfall equals or exceeds f_p; i.e., $f = f_p$ when $i \geq f_p$; but when $i < f_p$, $f < f_p$ and the actual infiltration rates are approximately equal to the rainfall rates.

The infiltration depends upon the intensity and duration of rainfall, weather (temperature), soil characteristics, vegetal cover, land use, initial soil moisture content (initial wet- ness), entrapped air and depth of the ground water table. The vegetal cover provides protec- tion against rain drop impact and helps to increase infiltration.

Methods of Determining Infiltration

The methods of determining infiltration are:

(i) Infiltrometers
(ii) Observation in pits and ponds
(iii) Placing a catch basin below a laboratory sample
(iv) Artificial rain simulators
(v) Hydrograph analysis

(i) Double-ring infiltrometer.

A double ring infiltrometer is shown in Fig. 3.7. The two rings (22.5 to 90 cm diameter) are driven into the ground by a driving plate and hammer, to penetrate into the soil uniformly without tilt or undue disturbance of the soil surface to a depth of 15 cm. After driving is over, any disturbed soil adjacent to the sides tamped with a metal tamper. Point gauges are fixed in the centre of the rings and in the annular space be- tween the two rings. Water is poured into the rings to maintain the desired depth (2.5 to 15 cm with a minimum of 5 mm) and the water added to maintain the original constant depth at regular time intervals (after the commencement of the experiment) of 5, 10, 15, 20, 30, 40, 60 min, etc. up to a period of atleast 6 hours is noted and the results are plotted as infiltration rate in cm/hr versus time in minutes as shown in Fig. 3.8. The purpose of the outer tube is to eliminate to some extent the edge effect of the surrounding drier soil and to prevent the water within the inner space from spreading over a larger area after

penetrating below the bottom of the ring.

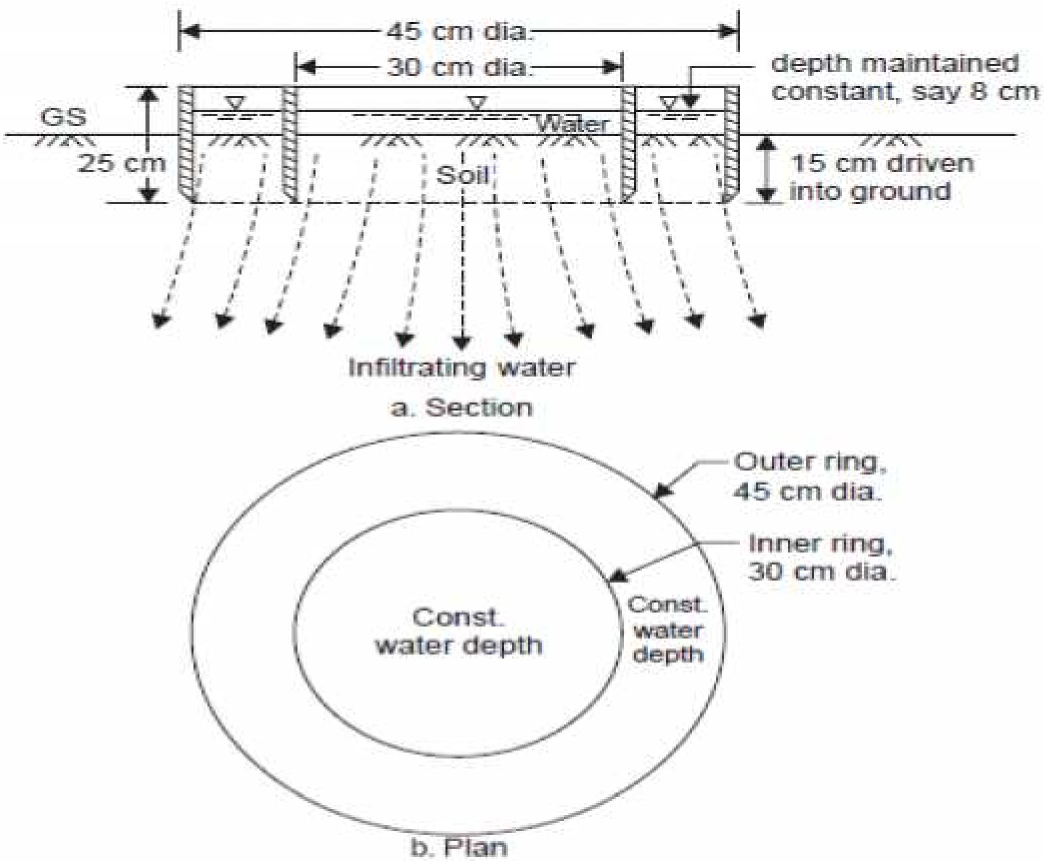


Fig. 3.7 Double ring infiltrometer

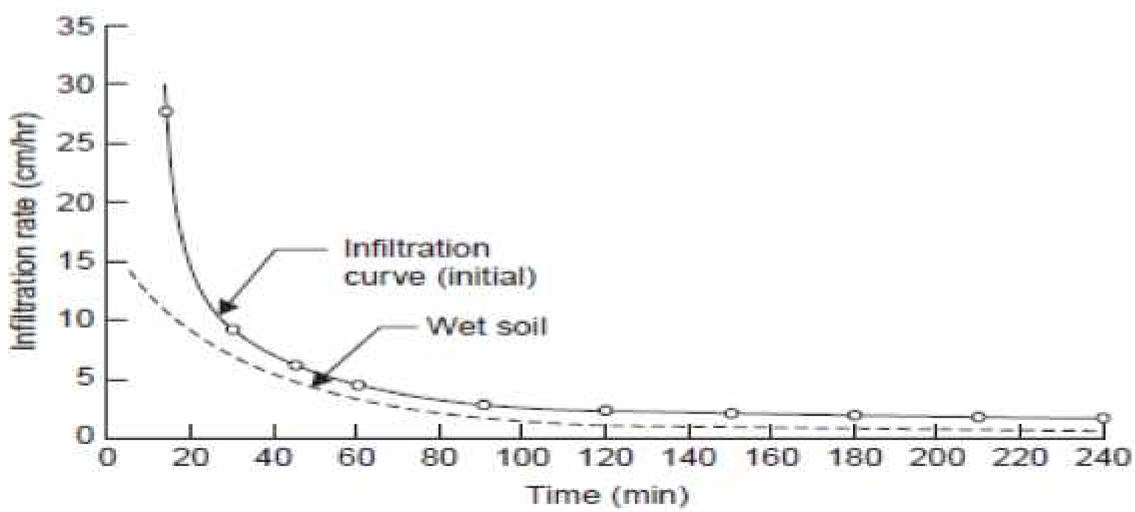


Fig. 3.8 Typical infiltration curve

Tube infiltrometer.

This consists of a single tube about 22.5 cm diameter and 45 to 60 cm long which is driven into the ground atleast to a depth up to which the water percolates during the experiment and thus no ~~lateral spreading of water can occur (Fig. 3.9). The water added into the tube at regular time~~ intervals to maintain a constant depth is noted from which the infiltration curve can be drawn.

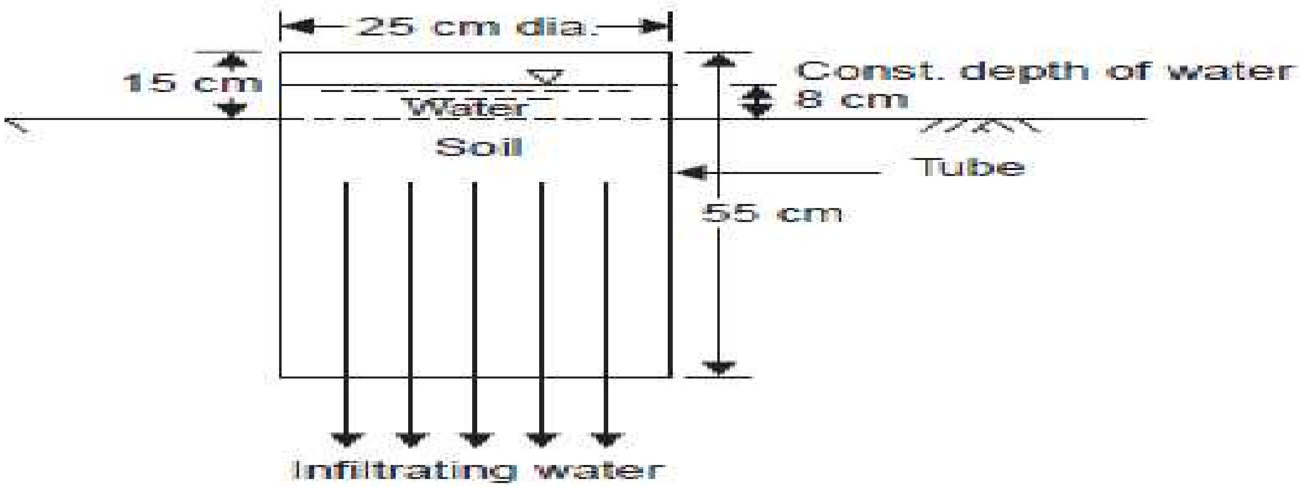


Fig. 3.9 Tube infiltrometer

INFILTRATION INDICES

The infiltration curve expresses the rate of infiltration (cm/hr) as a function of time. The area between the rainfall graph and the infiltration curve represents the rainfall excess, while the area under the infiltration curve gives the loss of rainfall due to infiltration. The rate of loss is greatest in the early part of the storm, but it may be rather uniform particularly with wet soil conditions from antecedent rainfall.

Estimates of runoff volume from large areas are sometimes made by the use of infiltra- tion indices, which assume a constant average infiltration rate during a storm, although in actual practice the infiltration will be varying with time. This is also due to different states of wetness of the soil after the commencement of the rainfall. There are three types of infiltration indices:

(I) ***φ-index***
(ii) W-index
(iii) f_{ave}***-index***

(i) φ-index*—The φ-index is defined as that rate of rainfall above which the rainfall volume equals the runoff volume. The φ-index is relatively simple and all losses due to infiltra- tion, interception and depression storage (i.e., storage in pits and ponds) are accounted for; hence,*

$$\varphi\text{-index} = \frac{\text{basin recharge}}{\text{duration of rainfall}}$$

provided $i > \varphi$ throughout the storm. The bar graph showing the time distribution of rainfall, storm loss and rainfall excess (net rain or storm runoff) is called a hyetograph, Fig. 3.12. Thus, the φ-index divides the rainfall into net rain and storm loss.

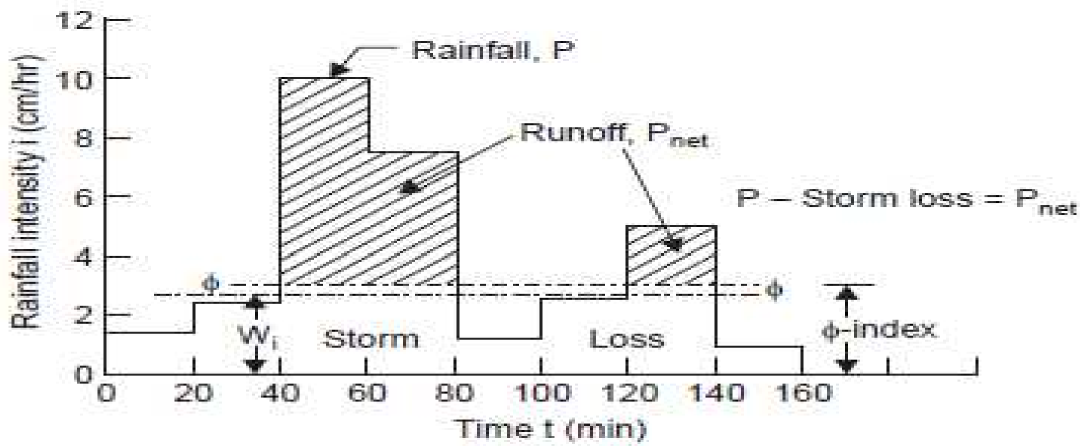


Fig. 3.12 Infiltration loss by ϕ-index

(ii) W-index—*The W-index is the average infiltration rate during the time rainfall intensity exceeds the infiltration capacity rate, i.e.,*

$$\text{W-index} = \frac{F_p}{t_R} = \frac{P - Q - S}{t_R}$$

where P = total rainfall

Q = surface runoff

S = effective surface retention

t_R = duration of storm during which $i > f_p$

F_p = total infiltration

The W-index attempts to allow for depression storage, short rainless periods during a storm and eliminates all rain periods during which $i < f_p$. Thus, the W-index is essentially equal to the φ-index minus the average rate of retention by interception and depression storage, i.e., $W < \varphi$.

Information on infiltration can be used to estimate the runoff coefficient C in computing the surface runoff as a percentage of rainfall i.e.,

$Q = CP$

$$C = \frac{i - W}{i}$$

(iii) f_{ave}-index— In this method, an average infiltration loss is assumed throughout the storm, for the period i > f.

WATER RESOURCES ENGINEERING

LECTURE NOTES

Module – II

Run off: Computation, factors affecting runoff, Design flood: Rational formula, Empirical formulae, Stream -flow: Discharge measuring structures, approximate average slope method, area-velocity method, stage-discharge relationship.

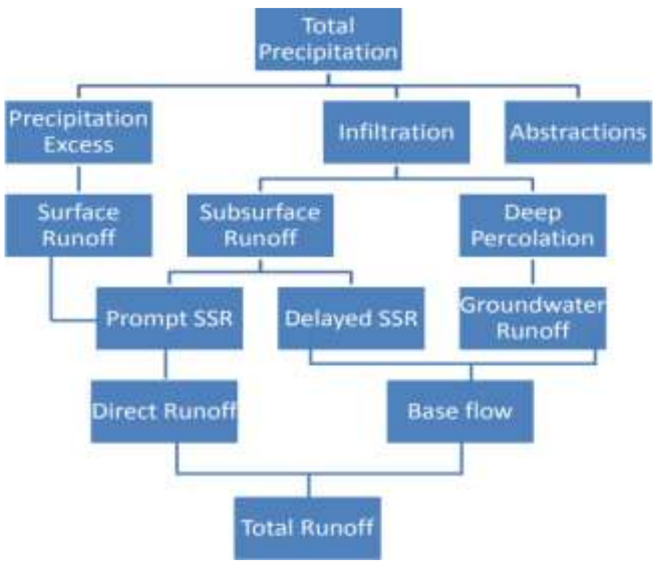
Hydrograph; Concept, its components, Unit hydrograph: use and its limitations, derivation of UH from simple and complex storms, S-hydrograph, derivation of UH from S-hydrograph. Synthetic unit hydrograph: Snyder's approach, introduction to instantaneous unit hydrograph (IUH).

Lecture Note 1

Runoff

Introduction

Runoff can be defined as the portion of the precipitation that makes it's way towards rivers or oceans etc, as surface or subsurface flow. Portion which is not absorbed by the deep strata. Runoff occurs only when the rate of precipitation exceeds the rate at which water may infiltrate into the soil.



Types of Runoff

• Surface runoff – Portion of rainfall (after all losses such as interception, infiltration, depression storage etc. are met) that enters streams immediately after occurring rainfall – After laps of few time, overland flow joins streams – Sometime termed prompt runoff (as very quickly enters streams)

• Subsurface runoff – Amount of rainfall first enter into soil and then flows laterally towards stream without joining water table – Also take little time to reach stream

• Base flow

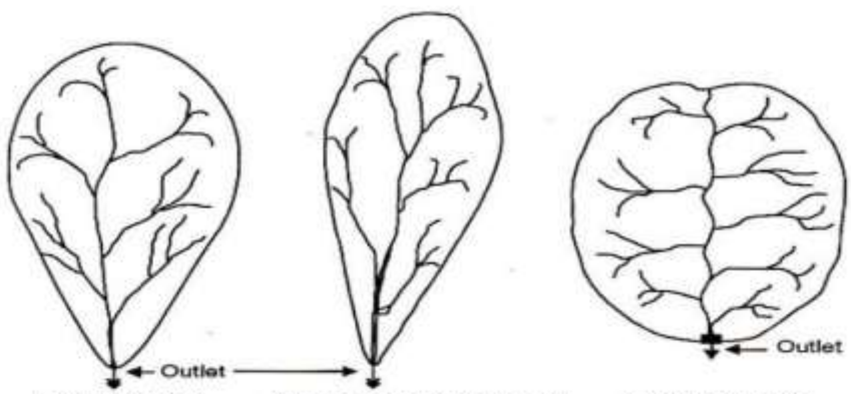
– Delayed flow

– Water that meets the groundwater table and join the stream or ocean

– Very slow movement and take months or years to reach streams

Factors affecting runoff

• Climatic factors
– Type of precipitation
• Rain and snow fall
– Rainfall intensity
• High intensity rainfall causes more rainfall
– Duration of rainfall
• When duration increases, infiltration capacity decreases resulting more runoff
– Rainfall distribution
• Distribution of rainfall in a catchment may vary and runoff also vary
• More rainfalls closer to the outlet, peak flow occurs quickly
• Direction of prevailing wind
– If the wind direction is towards the flow direction, peak flow will occur quickly
• Other climatic factors
– Temperature, wind velocity, relative humidity, annual rainfall etc. affect initial loss of precipitation and thereby affecting runoff
• Physiographic factors
– Physiographic characteristics of watershed and channel both
– Size of watershed
• Larger the watershed, longer time needed to deliver runoff to the outlet
• Small watersheds dominated by overland flow and larger watersheds by runoff
– Shape of watershed
• Fan shaped, fan shaped (elongated) and broad shaped



(a) Fan shaped (b) Fan shaped (Elongated) (c) Broad shaped

• Fan shaped – runoff from the nearest tributaries drained out before the floods of farthest tributaries. Peak runoff is less
• Broad shaped – all tributaries contribute runoff almost at the same time so that peak flow is more
– Orientation of watershed
• Windward side of mountains get more rainfall than leeward side
– Landuse
• Forest – thick layer of organic matter and undercover

– huge amounts absorbed to soil
– less runoff and high resistance to flow
• barren lands
– high runoff
– Soil moisture
• Runoff generated depend on soil moisture
– more moisture means less infiltration and more runoff
• Dry soil
– more water absorbed to soil and less runoff
– Soil type
• Light soil (sandy)
– large pores and more infiltration
• Heavy textured soils
– less infiltration and more runoff
– Topographic characteristics
• Higher the slope, faster the runoff
• Channel characters such as length, shape, slope, roughness, storage, density of channel influence runoff
- Drainage density

More the drainage density, runoff yield is more

Runoff Computation

• Computation of runoff depend on several factors

• Several methods available

– Rational method

– Cook's method

– Curve number method

– Hydrograph method

– Many more

Rational Method

• Computes peak rate of runoff

• Peak runoff should be known to design hydraulic structures that must carry it.

= Peak runoff rate (m3

/s) C = runoff coefficient

I = rainfall intensity (mm/h) for the duration equal to the time of

concentration A = Area of watershed (ha)

1.1.1 .1 Runoff coefficient

– Ratio of peak runoff rate to the rainfall intensity

– No units, 0 to 1

– Depend on landuse and soil type

– When watershed has many land uses and soil types, weighted average runoff coefficient is calculated

Runoff coefficient for Rational Method

S.No.	*Land use and topography*	*Soil type*		
		Sandy loam	*Clay and silt loam*	*Tight clay*
1.	Cultivated land			
	(i) Flat	0.30	0.50	0.60
	(ii) Rolling	0.40	0.60	0.70
	(iii) Hilling	0.52	0.70	0.82
2.	Pasture land			
	(i) Flat	0.10	0.30	0.40
	(ii) Rolling	0.16	0.36	0.55
	(iii) Hilling	0.22	0.42	0.60
3.	Forest land			
	(i) Flat	0.10	0.30	0.40
	(ii) Hilling	0.30	0.50	0.60
4.	Populated land			
	(i) Flat	0.40	0.55	0.65
	(ii) Rolling	0.50	0.65	0.80

3.1.1 .2 Time of concentration (Tc)

– Time required to reach the surface runoff from remotest point of watershed to its outlet

– At Tc all the parts of watershed contribute to the runoff at outlet

– Have to compute the rainfall intensity for the duration equal to time of concentration

– Several methods to calculate Tc

– Kirpich equation

L = Length of channel reach (m)
S = Average channel slope (m/m)

- Computation of rainfall intensity for the duration of Tc

- Assumptions of Rational Method

-Rainfall occur with uniform intensity at least to the Tc

–Rainfall intensity is uniform throughout catchment

- Limitations of Rational Method

–Uniform rainfall throughout the watershed never satisfied

–Initial losses (interception, depression storage, etc). are not considered

1.1.2 Cook's Method

Computes runoff based on 4 characteristics (relief, infiltration rate, vegetal cover and surface depression)

•Numerical values are assigned to each

Steps in calculation
•Step 1
–Evaluate degree of watershed characteristics by comparing with similar conditions

S. No.	Range	Numerical values assigned for runoff producing watershed's characteristics			
		Relief	Soil infiltration	Vegetal cover	Surface storage
1.	Low	(10 to 0) Land is relatively flat, average slope ranges from 0 to 5%.	(5) Infiltration rate is more than 2 cm/hour, soil contains high sand and loamy sand.	(5) About 9% of total area is covered under good vegetation either by forest or equivalent.	(5) Land consists of high surface depression, drainage system is not very well.
2.	Normal	(20 to 10) The land is rolling in shape and slope ranges from 5% to 10%.	(10) Infiltration rate varies from 0.75 to 2 cm/ hour, the soil is in normal and deep permeable nature.	(10) About 50% of total area is under good grass land or any other equivalent cover.	(10) Considerable depression storage, lakes, ponds and marshes are less than 2% of entire drainage system.
3.	High	(30 to 20) Lands are hilly in nature, average slope ranges from 10% to 30%.	(15) Infiltration rate ranges from 0.25 to 0.75 cm/ hour, the soil is relatively hard such as clay soil.	(15) Vegetal cover varies from poor to fair, less than 10% of total area is under grass cover.	(15) Surface depression is very low and area is well drained.
4.	Extreme	(40 to 30) Lands are steep and rugged terrain, slope ranges upto 30%.	(20) Infiltration rate is less.	(20) Land is bare, no effective grass cover.	(10) Surface depressions are negligible, drainage of land is very well, and no ponds or tanks are available.

Fig. (2) Numerical values for Cook's Method

Step 2

–Assign numerical value (W) to each of the characteristics

•Step 3

–Find sum of numerical values assigned

ΣW = total numerical value

R, I, V, and D are marks given to relief character, initial infiltration, vegetal cover and surface depression respectively

Step 4

–Determine runoff rate against ΣW using runoff curve (valid for specified geographical region and 10 year recurrence interval)

•Step 5

–Compute adjusted runoff rate for desired recurrence interval and watershed location

=P.R.F.S

= Peak runoff for specified geographical location and recurrence interval (m3/s)

P = Uncorrected runoff obtained from step 4

R = Geographic rainfall factor

F = Recurrence interval factor

S = Shape factor

1.1.1 Curve Number Method

• Calculates runoff on the retention capacity of soil, which is predicted by wetness status (Antecedent Moisture Conditions [AMC]) and physical features of watershed

• AMC - relative wetness or dryness of a watershed, preceding wetness conditions

• This method assumes that initial losses are satisfied before runoff is generated

Q = Direct runoff
P = Rainfall depth
S = Retention capacity of soil
CN = Curve Number
•CN depends on land use pattern, soil conservation type, hydrologic condition, hydrologic soil group

Land use pattern	*Treatment/practice adopted*	*Hydrologic condition*	*Hydrologic soil group*			
			A	*B*	*C*	*D*
Fallow-row crops	Straight row	—	77	86	91	94
		Poor	72	81	88	91
		Good	67	78	85	89
	Contoured + terraced condition	Poor	70	79	84	88
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured condition	Poor	63	74	82	85
		Good	63	75	83	87
	Contoured + terraced condition	Poor	61	72	79	82
		Good	59	70	78	81
Seeded	Straight row	Poor	66	77	85	89
Legumed		Good	55	69	78	83
Pasture land	Contoured condition	Poor	47	67	81	88
		Fair	25	59	75	83
		Good	6	35	70	79
Farm		Poor	45	66	77	83
Woodland		Fair	36	60	73	79
		Good	25	55	70	77
Hard surface			74	84	90	92
Farm steads			59	74	82	86
Meadow			330	58	71	78

Fig (2) Curve Numbers

Procedure
•Step 1
–Find value of CN using table

–Calculate S using equation

–Use equation and calculate Q (AMC II)

–Use correction factor if necessary to convert to other AMCs)
•Three AMC conditions

Factors for converting AMC II to AMC I or AMR III

CN – AMC II	Conversion Factor	
	AMC I	AMC III
10	0.40	2.22
20	0.45	1.85
30	0.50	1.67
40	0.55	1.50
50	0.62	1.40
60	0.67	1.30
70	0.73	1.21
80	0.79	1.14
90	0.87	1.07
100	1.00	1.00

Fig (3) Conversion Factor

AMC I –Lowest runoff generating potential –dry soil
•AMC II –Average moisture status
•AMC III –Highest runoff generating potential –saturated soil
•Soil A –low runoff generating potential, sand or gravel soils with high infiltration rates
•Soil B –Moderate infiltration rate, moderately fine to moderately coarse particles
•Soil C –Low infiltration rate, thin hard layer prevents downward water movement, moderately fine to fine particles
•Soil D –High runoff potential due to very low infiltration rate, clay soils

Classification of Streams

•Based on flow duration, streams are classified into

–Perennial

•Streams carry flow throughout the year

•Appreciable groundwater contribution throughout the year

–Intermittent

•Limited groundwater contribution

•In rainy season, groundwater table rises above stream bed

•Dry season stream get dried

–Ephemeral

•In arid areas

•Flow due to rainwater only

•No base flow contribution

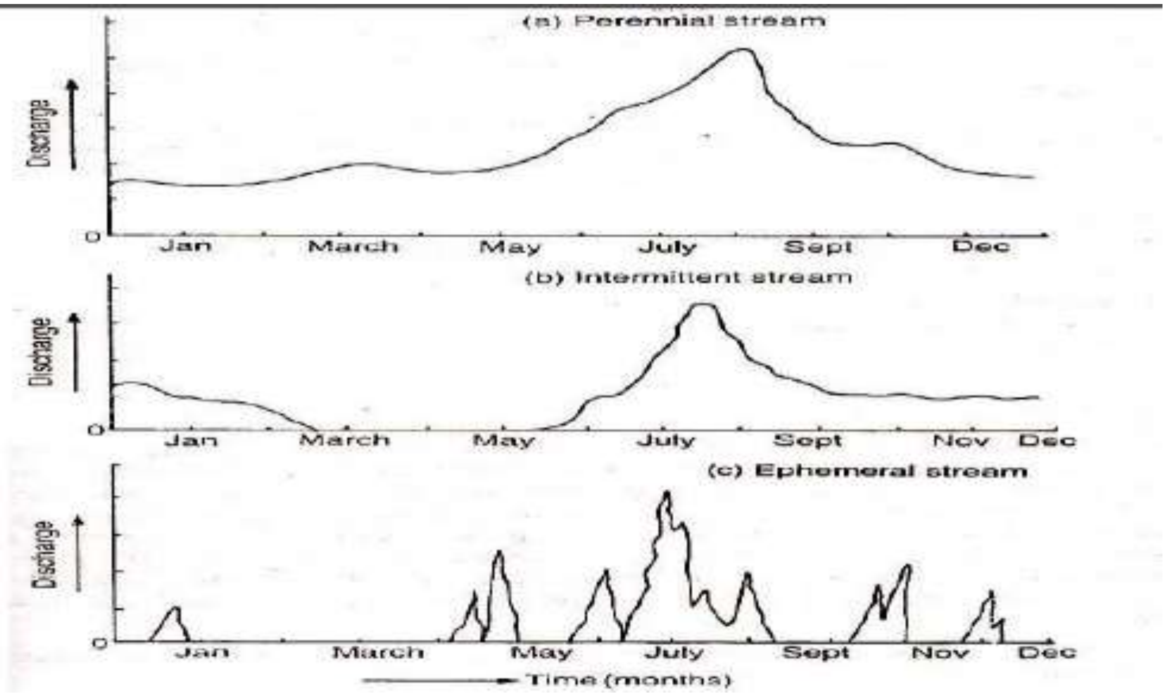


Fig (4) Classification of Stream

Flow Duration Curve

•Gives the variability of stream flow in a year

–Arrange stream flow data in descending order

–Assign rank number

–Calculate plotting position (Probability)

–Plot plotting position and discharge

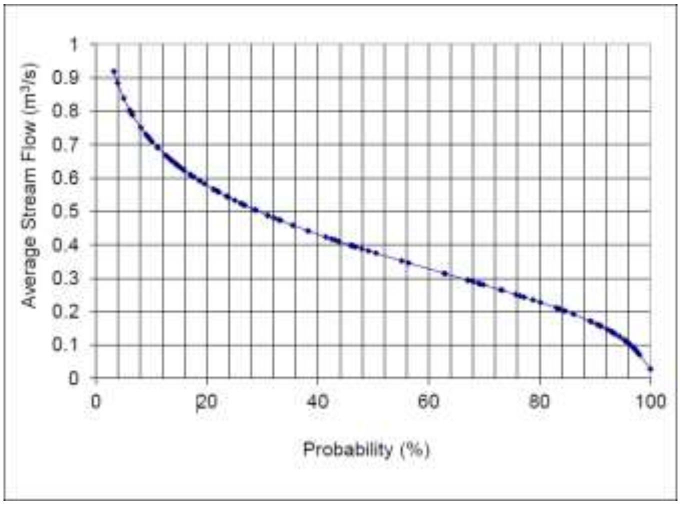


Fig (4) Flow Duration Curve

Characteristics of flow duration curve

–Steep slope –highly variable flow

–Flat slope –little variation in the flow

–Flat portion at top of curve –stream has large flood plain

–Flat portion at lower end –considerable baseflow

Uses of flow duration curve

–Discharge for any probability can be known

–Variation of flow within a year can be known

–Plan water resources projects

–Design of drainage structures

–Decide on flood control structures to be used

–Evaluate hydropower potential

–Determine sediment load carried by stream

Lecture Note 2

Streamflow Measurement

Introduction

Streamflow, or channel runoff, is the flow of water in streams, rivers, and other channels, and is a major element of the water cycle. It is one component of the runoff of water from the land to water bodies, the other component being surface runoff. Water flowing in channels comes from surface runoff from adjacent hill slopes, from groundwater flow out of the ground, and from water discharged from pipes. The discharge of water flowing in a channel is measured using stream gauges or can be estimated by the Manning equation. The record of flow over time is called a hydrograph. Flooding occurs when the volume of water exceeds the capacity of the channel.

Sources of Streamflow

Surface and subsurface sources: Stream discharge is derived from four sources: channel precipitation, overland flow, interflow, and groundwater.

- Channel precipitation is the moisture falling directly on the water surface, and in most streams, it adds very little to discharge. Groundwater, on the other hand, is a major source of discharge, and in large streams, it accounts for the bulk of the average daily flow.

- **Groundwater** enters the streambed where the channel intersects the water table, providing a steady supply of water, termed baseflow, during both dry and rainy periods. Because of the large supply of groundwater available to the streams and the slowness of the response of groundwater to precipitation events, baseflow changes only gradually over time, and it is rarely the main cause of flooding. However, it does contribute to flooding by providing a stage onto which runoff from other sources is superimposed.

- **Interflow** is water that infiltrates the soil and then moves laterally to the stream channel in the zone above the water table. Much of this water is transmitted within the soil itself, some of it moving within the horizons. Next to baseflow, it is the most important source of discharge for streams in forested lands. Overland flow in heavily forested areas makes negligible contributions to streamflow.

- In dry regions, cultivated, and urbanized areas, overland flow or surface runoff is usually a major source of streamflow. Overland flow is a stormwater runoff that begins as thin layer of water that moves very slowly (typically less than 0.25 feet per second) over the ground. Under intensive rainfall and in the absence of barriers such as rough ground, vegetation, and absorbing soil, it can mount up, rapidly reaching stream channels in minutes and causing sudden rises in discharge. The quickest response times between rainfall and streamflow occur in urbanized areas where yard drains, street gutters, and storm sewers collect overland flow and route it to streams straightaway. Runoff velocities in storm sewer piper can reach 10 to 15 feet per second.

Measurement

Streamflow is measured as an amount of water passing through a specific point over time. The units used in the United States are cubic feet per second, while in majority of other countries cubic meters per second are utilized. One cubic foot is equal to 0.028 cubic meters.

There are a variety of ways to measure the discharge of a stream or canal. A stream gauge provides continuous flow over time at one location for water resource and environmental management or other purposes. Streamflow values are better indicators than gage height of conditions along the whole river. Measurements of streamflow are made about every six weeks by United States Geological Survey (USGS) personnel. They wade into the stream to make the measurement or do so from a boat, bridge, or cableway over the stream. For each streamgaging station, a relation between gage height and streamflow is determined by simultaneous measurements of gage height and streamflow over the natural range of flows (from very low flows to floods). This relation provides the current condition streamflow data from that station.[2] For purposes that do not require a continuous measurement of stream flow over time, current meters or acoustic Doppler velocity profilers can be used. For small streams — a few meters wide or smaller — weirs may be installed.

2.2 Methods of forecasting streamflow

For most streams especially those with small watershed, no record of discharge is available. In that case, it is possible to make discharge estimates using the rational method or some modified version of it. However, if chronological records of discharge are available for a stream, a short term forecast of discharge can be made for a given rainstorm using a hydrograph.

Unit Hydrograph Method. This method involves building a graph in which the discharge generated by a rainstorm of a given size is plotted over time, usually hours or days. It is called the unit hydrograph method because it addresses only the runoff produced by a particular rainstorm in a specified period of time- the time taken for a river to rise, peak, and fall in response to a storm. Once rainfall-runoff relationship is established, then subsequent rainfall data can be used to forecast streamflow for selected storms, called standard storms. A standard rainstorm is a high intensity storm of some known magnitude and frequency. One method of unit hydrograph analysis involves expressing the hour by hour or day by day increase in streamflow as a percentage of total runoff. Plotted on a graph, these data from the unit hydrograph for that storm, which represents the runoff added to the prestorm baseflow. To forecast the flows in a large drainage basin using the unit hydrograph method would be difficult because in a large basin geographic conditions may vary significantly from one part of the basin to another. This is especially so with the distribution of rainfall because an individual rainstorm rarely covers the basin evenly. As a result, the basin does not respond as a unit to a given storm, making it difficult to construct a reliable hydrograph.

Magnitude and frequency method. For large basins, where unit hydrograph might not be useful and reliable, the magnitude and frequency method is used to calculate the probability of recurrence of large flows based on records of past years' flows. In United States, these records are maintained by the Hydrological Division of the U.S. Geological Survey for most rivers and large streams. For a basin with an area of 5000 square miles or more, the river system is typically gauged at five to ten places. The data from each gauging station apply to the part of the basin upstream that location. Given several decades of peak annual discharges for a river, limited projections can be made to estimate the size of some large flow that has not been experienced during the period of record. The technique involves projecting the curve (graph line) formed when peak annual discharges are plotted against their respective recurrence intervals. However,

in most cases the curve bends strongly, making it difficult to plot a projection accurately. This problem can be overcome by plotting the discharge and/or recurrence interval data on logarithmic graph paper. Once the plot is straightened, a line can be ruled drawn through the points. A projection can then be made by extending the line beyond the points and then reading the appropriate discharge for the recurrence interval in question.

Categorisation Of Streamflow Measurement

Stream flow techniques are broadly classified into two categories:-

(1) Direct determination of discharge

(2) Indirect determination of discharge

Direct Method

• Direct determination of stream discharge measurement includes :-
(a) Area velocity method
(b) Dilution techniques
(c) Electromagnetic method

(d) Ultrasonic method

Indirect Method

• Indirect method of stream discharge measurement includes :-

(a) Slope area method

(b) Hydraulic structures

- Continuous measurement of stream discharge is very difficult. As a rule direct measurement of discharge is a very time consuming and costly procedure.
- Hence a two step procedure is followed.
- At first the discharge in a given stream is related to the elevation of the water surface(stage) through avseries of careful measurement.
- In the next step, the stage of the stream is observedvroutinely in a relatively inexpensive manner and thevdischarge is estimated by using the previouslyvdetermined stage-discharge relationship.
- This method of discharge determination of streams is adopted universally.

Measurement Of Stage

• The stage of a river is defined as its water surface elevation measured above a datum(Mean Sea Level or any datum connected independently to MSL)
• For the measurement of stage we have:-
(1). Manual gauges
(2). Automatic stage recorder
• Under Manual gauges we have
(a). Staff gauge
(b). Wire gauge

Stage Data

• The stage data is often represented in the form of a plot of stage against chronological time.
• This is popularly known as stage hydrograph
• Stage data is of utmost importance in design of hydraulic structures, flood warning and flood protection work.
Abscissa- Time
Ordinate- Stage

Measurement Of Velocity

• For the accurate determination of velocity in a stream we have a mechanical device known as CURRENT METER.
• It is the most commonly used instrument in Hydrometry to measure the velocity at a point in the flow cross-section.
• It essentially consists of a rotating element which rotates due to the reaction of the stream current with an angular velocity proportional to the stream velocity.
• Robert Hooke(1663) invented a propeller type current meter for traversing the distance covered by ship.
• Later on it was Henry(1868) who invented present day cup- type instrument and the electrical make-and-break mechanism.

Types Of Current Meter

• VERTICAL AXIS METERS
• HORIZONTAL AXIS METERS
A current meter is so designed that its rotation speed varies linearly with the stream velocity v .
• A typical relationship is:-

$$v = a\,N_s + b$$

Where v= stream velocity at the instrument location.
N_s= no of revolutions per second of the meter
a, b= constants of the meter.

Calibration Equation

• Now we need to find out the relation between the stream velocity and revolutions per second of the meters which is nothing but the calibration equation.
• The calibration equation is unique to each instrument .
• It is determined by towing the instrument in a special tank
• A towing tank is a long channel containing still water with arrangements for moving a carriage longitudinally over its surface at constant speed.
The instrument to be calibrated is mounted on the carriage with the rotating element immersed to a specified depth in the water body in the tank.
• The carriage is then towed at a predetermined constant speed(v) and the corresponding avg value of revolutions per second is determined.
• In India we have an excellent towing tank facilities for calibration of current meters at CWPRS(Central Water And Power Research Station) and IIT MADRAS.

Value Of Velocity For Field Use

• The velocity distribution in a stream across a vertical section is logarithmic in nature.
• The velocity distribution is given by

$$v = 5.75v^* \log10 (30y/k_s)$$

• V= velocity at a point y above the bed
• V* = shear velocity
• K_s = equivalent sand grain roughness
• In order to determine the accurate avg velocity in a vertical section one has to measure the velocity at large no of points which is quite time consuming. So certain specified procedure have been evolved.
• For the streams of depth 3.0 m the velocity measured at 0.6 times the depth of flow below the water surface is taken as the average velocity.(single point observation model)

$$V(avg) = v_{0.6}$$

• For the depth between 3.0m to 6.0 m we have

$$V(avg) = (v_{0.2} + v_{0.8})/2$$

•For the river having flood flow we have

$$v(avg) = k^*v_s$$

where v_s(surface velocity) and k(reduction coefficient)
Value of k lies between 0.85 to 0.95

Sounding Weight

• Current meter is weighted down by lead weight called sounding weight.
• It is connected to the current meter with a hangar bar and pin assembly.
• These weights are of steramlined shapes with a fin in the rear
• The minimum weight of sounding weight is estimated as

$$w = 50^*v_{avg}^*d$$

Where d= depth of flow at vertical
v_{avg} = avg velocity

Vertical Axis Meters

• This instrument consist of a series of conical cups mounted around a vertical axis.
• The cups rotate in a horizontal plane .
• The cam attached to the vertical axial spindle records generated signals proportional to the revolutions of the cup assembly.
• Range of velocity is **0.155m/s to 2.0m/s**.
• **It can not be used in situations where there are appreciable vertical components of velocities.**
• **Price current meter and Gurley current meter** are some of its type.

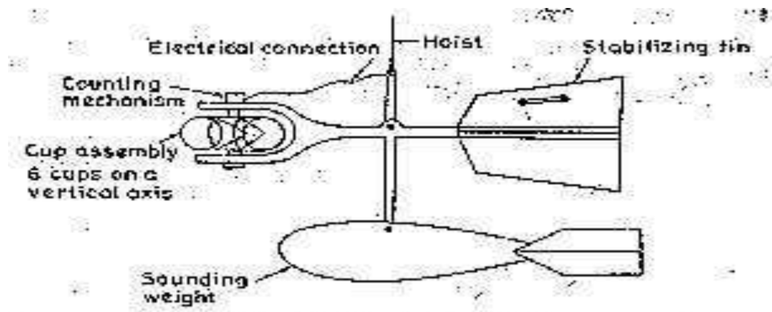


Fig (1) Vertical Axis Current Meter

Horizontal Axis Meters

• This instrument consist of a propeller mounted at the end of horizontal shaft .
• The propeller **diameter is in the range of 6 to 12cm**
• It can register velocities from **0.155m/s to 4.0m/s.**
• This meter is not affected by **oblique flows of as much as 150 .**
• **Ott, Neyrtec, Watt** type meters are typical instruments under this kind.

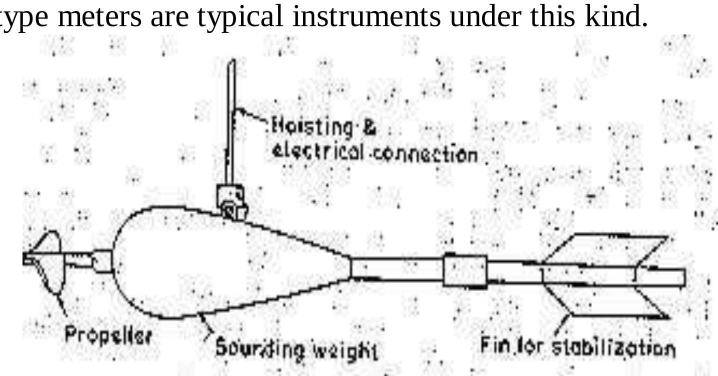


Fig (2) Propeller type Current Meter

Area Velocity Method

• This method of discharge measurement consists essentially of the area of cross section of the river at a selected section called the **gauging site** and measuring the velocity of flow through the cross sectional area.
• The gauging site must be selected with care to ensure that the stage discharge curve is reasonably constant over a long period over a few years. Toward the statement given in the previous slide the following criteria are adopted
• **(a)** The stream should have a well defined cross section which does not change in various seasons.
• **(b)** It should be easily accessible all through the year.
• **(c)** The site should be in straight, stable reach.
• **(d)** The gauging site should be free from backwater effects in the channel

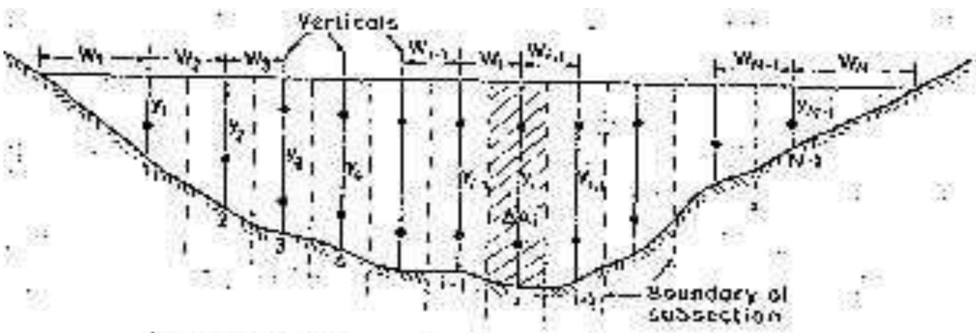


Fig (3) Stream section for area velocity method

➢ How basically the depth of a river is determined?

• . At the selected site, the section line is marked off by permanent survey markings and the cross section is determined.

Case 1 – when the stream depth is small.

• (The depth at various locations are measured by sounding rods or sounding weights.)

Case 2 – when the stream depth is large or when result is needed with higher accuracy.

• (An instrument named echo-depth recorder is used. In this, a high frequency sound wave is sent down by a transducer kept immersed in water surface and the echo reflected by the bed is recorded by the same transducer. By comparing the time interval between the transmission of the signal and the receipt of the echo the distance is measured and is shown . It is particularly advantageous in high velocity streams.)

➢ How cross section of a river is determined?

• The cross section is considered to be divided into large number of subsections by verticals. The average velocity in these subsections are measured by current meters or by floats.
• It is quite obvious that the accuracy of discharge estimation increases with the increase in number of subsections.
• The following are some of the guidelines to select the number of segments
• (a) the segment width should not be greater than 1/15 or 1/20 of the width of the river.
• (b) the discharge in each segment should be smaller than the 10% of the total discharge.
• (c) the difference of velocities in adjacent segments should not be more than 20%
• This is also called Standard Current Meter Method.

Moving Boat Method

• Discharge measurement of large alluvial rivers, such as the ganga by the standard current meter method is very time consuming even the flow is low or moderate.
• When the river is spate, it is impossible to use the standard current meter technique due to the difficulty in keeping the boat stationary on the fast moving surface of the stream .
• It is such circumstances that the moving boat techniques prove very helpful
• In this method, a special propeller- type current meter which is free to move about a vertical axis is towed in a boat at a velocity v_b at right angle to the stream flow . If the flow velocity is v_f, the meter will align itself in the direction of the resultant velocity vg making an angle θ with the direction of the boat . The meter will register the velocity v_r . If v_b normal to v_f then:-

$$v_b = v_r \cos\theta \text{ and } v_f = v_r \sin\theta$$

• If the time of transit between two verticals is Δt then the width between the two verticals is

$$\mathbf{w = v_b \, \Delta t}$$

• The flow in the sub-area between two verticals i and i+1 where the depths are y_i and y_{i+1} respectively by assuming the current meter to measure the average velocity in the vertical is :-

$$\Delta Q_i = [\{y_i + y_{i+1}\}/2] w_{i+1} v_f$$

$$\Delta Q_i = [\{y_i + y_{i+1}\}/2] v^2{}_r \sin\theta \cos\theta \, \Delta t$$

• Summation of the partial discharges ΔQi over the whole width of the stream gives the stream discharge.

Dilution Technique

• Also known as chemical method.
• Depends on the continuity principle. This principle is applied to a tracer which is allowed to mix completely with the flow.
• Two methods of dilution technique:-
(a) sudden injection method/ gulp / integration method.
(b) constant rate injection method/plateau gauging
NOTE
• Dilution method of gauging is based on the assumption of steady flow. If the flow is unsteady and the flow changes appreciably during gauging. There will be a change in the storage volume in the reach and the steady state continuity equation is not valid.

Constant Rate Injection Method

• It is one particular way of using the dilution principle by injecting the tracer of a concentration c_1 at constant rate Q_1 at section1. At section 2 the concentration gradually rises from the background value of c_0 at time t_1 to a constant value c_2 .so at the steady state, the continuity equation for the tracer is :-

$$Q_t C_1 + QC_0 = (Q+Q_t)C_2$$

$$Q = Q_t(C_1 - C_2)/(C_2 - C_0)$$

➢ IMPORTANT POINTS REGARDING TRACERS

• Tracers are of three main types:-
(a) chemicals (sodium chloride, sodium dichromate)
(b) fluorescent dyes (rhodamine- WT, sulpho-rhodamine)
(c) radioactive materials(bromine-82, sodium-22,iodine-132)
Tracers should ideally follow the following property:-
• 1 non-toxic
• 2 not be very expensive
• 3 should be capable of being detected even at a very small concentrations
• 4 should not get absorbed by the sediments or vegetation.
• 5 it should be lost by evaporation.
• 6 should not chemically react with any of the surfaces like channel boundary or channel beds

Length Of Reach

• It is the distance between the dosing section and sampling section which should be large enough to have the proper mixing of the tracer with the flow.

• The length depends on the geometric dimensions of the channel cross section , discharge and turbulence levels.
• Empirical formula suggested by **RIMMAR (1960**) for the estimation of the mixing length
.

L = 0.13B2C{0.7C+2(g1/2)}/gd

• L= MIXING LENGTH, B= AVG WIDTH, d= AVG DEPTH, C= CHEZY'S CONSTANT g = ACCLN DUE TO GRAVITY

The dilution method has the major advantage that the discharge is estimated directly in an absolute way. It is particularly attractive for small streams such as mountainous rivers
• It can be used occasionally for checking the calibration , stage discharge ,curves etc obtained by other methods.

Ultrasonic Methods

• It is essentially an area velocity method.
• The average velocity is only measured using ultrasonic signals.
• Reported by SWENGEL(1955)

Indirect Methods

• These category include those methods which make use of the relationship between the flow discharge and the depths at specified locations.
• Two broad categories under this method is:-
(a) flow measuring structures
(b) slope area method

Flow Measuring Structures

• Structures like notches, weir, flumes and sluice gates for flow measurements in hydraulic laboratories are well known.
• These conventional structures are used in the field conditions but their use is limited by the ranges of head, debris or sediment load of the stream and the backwater effects produced by the streams.

The basic principle governing the use of these structure is that these structure produce a unique control section in the flow. At these structure the discharge Q is the function of the water surface elevation measured from the specified datum.

$Q = f(H)$

H= water surface elevation measured from the specified datum

e.g. $Q = KH^n$ where[$K = 2/3\ c_d\ b(2g)^{1/2}$] used basically for weirs. K and n are system constants.

•The above red marked equation is valid as long the downstream water level is below a certain limiting water level known as modular limit.
•The flows which are independent of downstream water level are known as free flows.
• If the tail water conditions do affect the flow then the flow is called drowned flow /submerged flow.
• Discharges under drowned conditions are estimated by VILLEMONTE FORMULA.

$Q_s = Q_1\ [1-(H_2 / H_1)^n]^{0.85}$

Q_s = submerged discharge

Q_1 = free flow discharge under H1

H_1 = Upstream water surface elevation measured above weir crest

H_2 = downstream water surface elevation measured above weir crest
n = for rectangular weir n = 1.5

CATEGORIZATION OF HYDRAULIC STRUCTURE

(a) thin plate structures

(b) long base weirs[broad crested structures]

(c) flumes[made of concrete, masonry, metal sheets etc]

SLOPE AREA METHOD

• Manning's formula is used to relate depth at either section with the discharge.

• Knowing the water surface elevation at the two section, it is required to estimate the discharge. Applying the energy equation to sections 1 and 2,

• $Z_1 + Y_1 + \{V_1{}^2 / 2g\} = Z_2 + Y_2 + \{V_2{}^2 /2g\} + h_L$

• $h_L = h_e + h_f$ where h_e = eddy loss and h_f = frictional loss.

• $h_f = (h_1 - h_2) + \{(v_1{}^2 /2g) - (v_2{}^2 /2g)\} - h_e$

• If L = Length of the reach then $h_f / L = S_f = Q^2 / K^2$ = Energy slope

• K= conveyance of the channel =(1/n)A(R2/3)

• n= manning's roughness coefficients

• In non uniform flow $k = \{k_1 k_2\}^{1/2}$

• $H_e = K_e [(v_1{}^2/2g) - (v_2{}^2 /2g)]$ where K_e = eddy loss coefficient .

Stage Discharge Relationship

• The stage discharge relationship is also known as **rating curve**.

• The measured value of discharges when plotted against the corresponding stages gives relationship that represents the integrated effects of a wide range of channel and flow parameters.

• The combined effects of these parameters is known as **control**.

• If the (G-Q) relationship for a gauging section is constant and does not change with time, the control is called **permanent**.

• If it changes with time , it is called **shifting control**

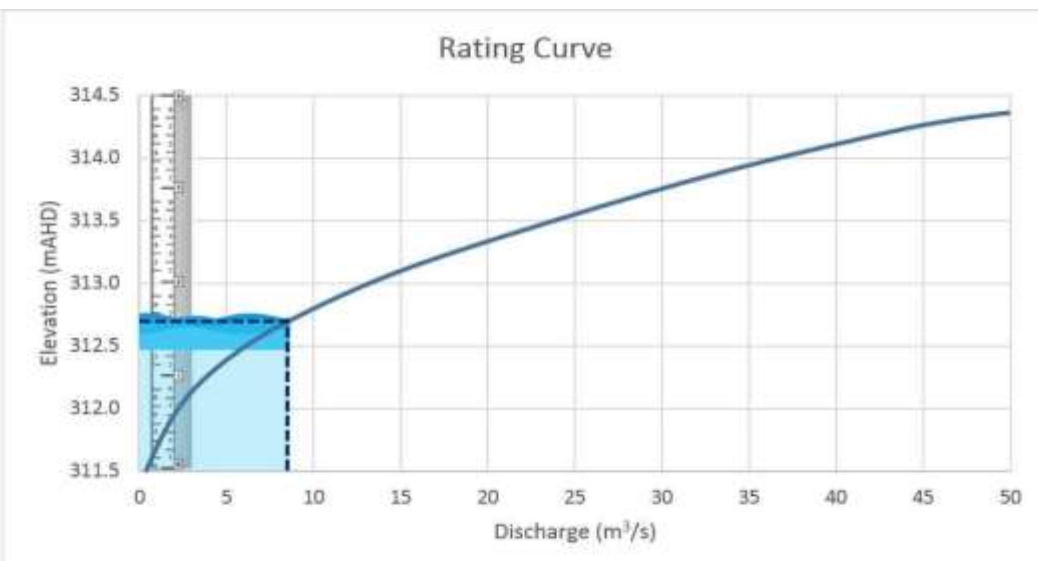


Fig (5) Rating Curve

Permanent Control

- Non alluvial rivers basically exhibit permanent control.
- $Q= Cr (G-a) \beta$
- Q= stream flow discharge
- Cr and β are rating curve constant
- a = constant representing the gauge reading corresponding to zero discharge.

NOTE

- Cr and β need not be the same for the full range of stages, Best possible way to find the value of cr and β(Rating curve constant)
- It is best obtained by the least square errorvmethod.
- Mathematically one can represent it in :-
- $\log Q = \beta \log(G-a) + \log cr$
- $Y = \beta X + b$

$\beta = \{N(\Sigma XY)-(\Sigma X)(\Sigma Y)\}/\{N(\Sigma X2)- (\Sigma X)2\}$

$b= \{\Sigma Y - \beta(\Sigma X)\}/N$

•Pearson product moment correlation coefficient

. $r = N(\Sigma XY)- (\Sigma X)(\Sigma Y)/[\{N(\Sigma X2)-(\Sigma X)2\}\{N(\Sigma Y)2-(\Sigma Y)2]1/2$

If r = 0.6 to 1.0 , it is generally taken as a good Correlation

Shifting Control

- The control that exists in between stage discharge relationship changes due to:-
- (1). Changing characteristics caused by the weed growth, dredging or channel encroachment
- (2).aggradation or degradation phenomenon in an alluvial channel.
- (3). Variable backwater effects affecting the gauging station.
- (4). Unsteady flow effects of a rapidly changing stage.

Lecture Note 3

Hydrograph

Introduction

- A **hydrograph** is a graph showing the rate of flow (discharge) versus time past a specific point in a river, channel, or conduit carrying flow. The rate of flow is typically expressed in cubic meters or cubic feet per second (cms or cfs). It can also refer to a graph showing the volume of water reaching a particular outfall, or location in a sewerage network. Graphs are commonly used in the design of sewerage, more specifically, the design of surface water sewerage systems and combined sewers.

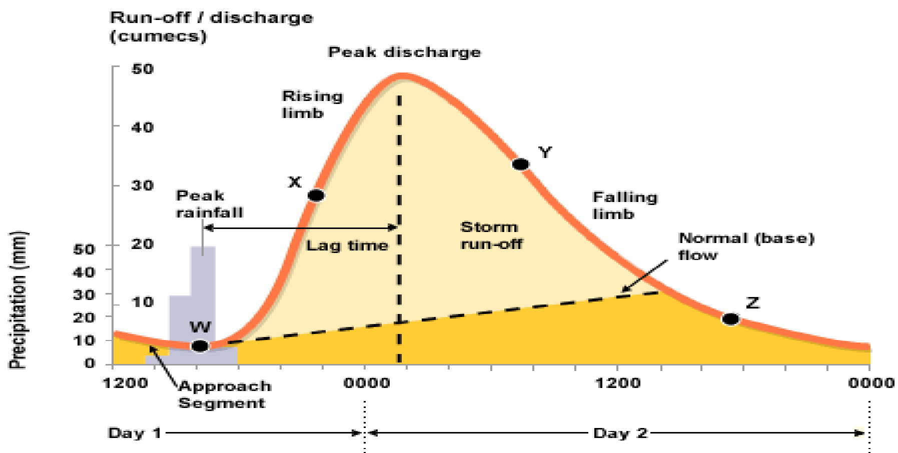


Fig (1) Hydrograph

Its Components

- **Discharge:** the rate of flow (volume per unit time) passing a specific location in a river, or other channel. The discharge is measured at a specific point in a river and is typically time variant.
- **Rising limb:** The rising limb of the hydrograph, also known as concentration curve, reflects a prolonged increase in discharge from a catchment area, typically in response to a rainfall event.
- **Peak discharge:** the highest point on the hydrograph when the rate of discharge is greatest.
- **Recession (or falling) limb:** The recession limb extends from the peak flow rate onward. The end of stormflow (a.k.a. quickflow or direct runoff) and the return to groundwater-derived flow (base flow) is often taken as the point of inflection of the recession limb. The recession limb represents the withdrawal of water from the storage built up in the basin during the earlier phases of the hydrograph.
- **Lag time:** the time interval from the center of mass of rainfall excess to the peak of the resulting hydrograph.
- **Time to peak:** time interval from the start of the resulting hydrograph.

Unit Hydrograph

- A unit hydrograph (UH) is the hypothetical unit response of a watershed (in terms of runoff volume and timing) to a unit input of rainfall. It can be defined as the direct runoff hydrograph (DRH) resulting from one unit (e.g., one cm or one inch) of effective rainfall occurring uniformly over that watershed at a uniform rate over a unit period of time. As a UH is applicable only to the direct runoff component of a hydrograph (i.e., surface runoff), a separate determination of the baseflow component is required.
- A UH is specific to a particular watershed, and specific to a particular length of time corresponding to the duration of the effective rainfall. That is, the UH is specified as being the 1-hour, 6-hour, or 24-hour UH, or any other length of time up to the time of concentration of direct runoff at the watershed outlet. Thus, for a given watershed, there can be many unit hydrographs, each one corresponding to a different duration of effective rainfall.
- The UH technique provides a practical and relatively easy-to-apply tool for quantifying the effect of a unit of rainfall on the corresponding runoff from a particular drainage basin. UH theory assumes that a watershed's runoff response is linear and time-invariant, and that the effective rainfall occurs uniformly over the watershed. In the real world, none of these assumptions are strictly true. Nevertheless, application of UH methods typically yields a reasonable approximation of the flood response of natural watersheds. The linear assumptions underlying UH theory allows for the variation in storm intensity over time (i.e., the storm hyetograph) to be simulated by applying the principles of superposition and proportionality to separate storm components to determine the resulting cumulative hydrograph. This allows for a relatively straightforward calculation of the hydrograph response to any arbitrary rain event.
- An instantaneous unit hydrograph is a further refinement of the concept; for an IUH, the input rainfall is assumed to all take place at a discrete point in time (obviously, this isn't the case for actual rainstorms). Making this assumption can greatly simplify the analysis involved in constructing a unit hydrograph, and it is necessary for the creation of a geomorphologic instantaneous unit hydrograph.

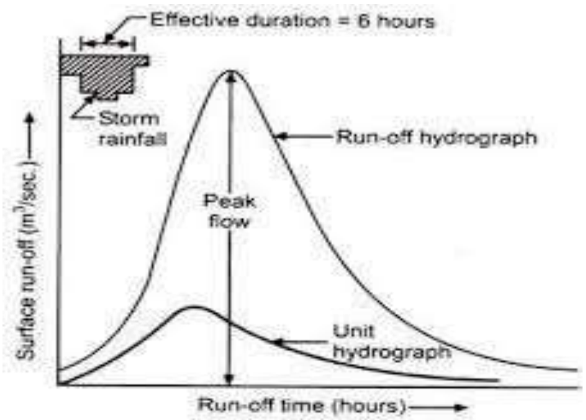


Fig (2) Unit Hydrograph

Basic Assumptions Of UH

(i) The effective rainfall is uniformly distributed over the entire drainage basin.
(ii) The effective rainfall occurs uniformly within its specifier duration.
This requirement calls for selection of storms of so small a duration which would generally produce an intense and nearly uniform effective rainfall and would produce a well defined single peak of hydrograph of short time base. Such a storm can be termed as ―unit storm‖.
(iii) The effective rainfalls of equal (unit) duration will produce hydrographs of direct runoff having same or constant time base.
(iv) The ordinates of the direct runoff hydrographs having same time base (i.e., hydrographs due to effective rainfalls of different intensity but equal duration) are directly proportional to the total amount of direct runoff given by each hydrograph. This important assumption is called principle of linearity or proportionality or superposition.
(v) The hydrograph of runoff from a given drainage basin resulting, from a given pattern of rainfall reflects all the combined physical characteristics of the basin. In other words the hydrograph of direct runoff resulting from a given pattern of effective rainfall will remain invariable irrespective of its time of occurrence. This assumption is called principle of time invariance.

Limitations

(i) In theory, the principle of unit hydrograph is applicable to a drainage basin of any size. In practice, however, uniformly distributed effective rainfall rarely occurs on large areas. Also on large areas effective rainfall is very rarely uniform at all locations, within its specified duration. Obviously bigger the area of the drainage basin lesser will be the chances of fulfilling the assumptions enunciated above. The limiting size of the drainage basin is considered to be 3000 km^2. Beyond it the reliability of the unit hydrograph method diminishes.
When the area of the drainage basin exceeds a few thousand km^2. The catchment has to be divided into sub-basins and the unit hydrographs developed for each sub-basin. The flood discharge at the basin outlet can then be estimated by combining the sub- basin floods adopting flood routing procedure.
(ii) The unit hydrograph method cannot be applied when appreciable portion of storm precipitation falls as snow because snow-melt runoff is governed mainly by temperature changes.
(iii) Also when snow covered area in the drainage basin is significant the unit hydrograph method becomes inapplicable. The reason is that the storm rainfall gets mixed up with the snow pack and may produce delayed runoff differently under different conditions of snow pack.
(iv) The physical basin characteristics change with seasons, man-made structures in the basin, conditions of flow etc. Obviously the principle of time invariance is really valid only when the time and condition of the drainage basin are specified.
(v) It is commonly seen that no two rain storms have same pattern in space and time. But it is not practicable to derive separate unit hydrograph for each possible time- intensity pattern. Therefore, in addition to limiting drainage basin area up to 5000 km^2 if storms of shorter duration say 1/3 to 1/4 of peaking time are selected it is seen that the runoff patterns do not vary drastically.

(vi) The principle of linearity is also not completely valid. This is so because due to variability in proportion of surface, subsurface and groundwater runoff components during smaller and larger storms of same duration, the maximum ordinate (peak) of the unit hydrograph derived from smaller storm is smaller than the one derived from larger storm. Obviously the character and duration of recession limb which is a function of the peak flow will also be different. When appreciable non-linearity is seen to exist it is necessary to use derived unit hydrographs only for reconstructing events of similar magnitude.

(vii) The unit hydrograph can be used theoretically to construct a flood hydrograph resulting from a storm having same unit duration. Obviously it necessitates construction of several unit hydrographs to cover different durations of storms. In practice however it is seen that a tolerance of ± 25% in unit hydrograph duration is acceptable. Thus a 2 hour unit hydrograph can be applied to storms of 1.5 to 2.5 hours duration.

Advantages of Unit Hydrograph Theory:

The limitation to the theory of unit hydrograph can be overcome to a large extent by remaining within the various ranges and restrictions indicated above.

The unit hydrograph theory has several advantages to its credit which can be summarised as below:

(i) Flood hydrograph can be calculated with the help of very short record of data.

(ii) In addition to peak flow unit hydrograph also gives total volume of runoff and its time distribution.

(iii) The unit hydrograph procedure can be computerised easily to facilitate calculations.

(iv) It is very useful in checking the reliability of flows obtained by using statistical methods.

Derivation Of Unit Hydrographs

1. A number of isolated storm hydrographs caused by short spells of rainfall excess, each of approximately the same duration (0.9 to 1.1D h) are selected from a study of continuously gauged runoff of the stream

2. For each of these surface runoff hydrographs, the base flow is separated

3. The area under DRH is evaluated and the volume of direct runoff obtained is divided by the catchment area to obtain the depth of ER

4. The ordinates of the various DRHs are divided by the respective ER values to obtain the ordinates of the unit hydrograph

- Flood hydrographs used in the analysis should be selected so as to meet the following desirable features with respect to the storms responsible for them:

1. The storms should be isolated storms occurring individually

2. The rainfall should be fairly uniform during the duration and should cover the entire catchment area

3. The duration of rainfall should be 1/5 to 1/3 of the basin lag

4. The rainfall excess of the selected storm should be high (A range of ER values of 1.0 to 4.0 cm is preferred)

• A number of unit hydrographs of a given duration are derived as mentioned above and then plotted

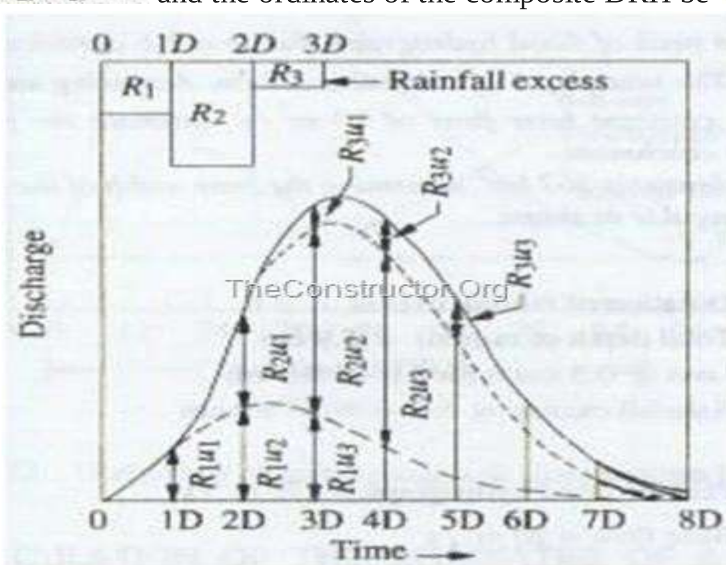
• Because of spatial and temporal variations in rainfall and due to deviations of the storms from the assumptions in the unit hydrograph theory, the various unit hydrographs developed will not be exactly identical

• In general, the mean of these curves is adopted as the unit hydrograph of the given duration for the catchment
• The average of the peak flows and the time to peaks are computed first
• Then a mean curve of best fit (by eye judgment) is drawn through the averaged peak to close on an averaged base length
• The volume of the DRH is determined and any departure from unity is corrected by adjusting the peak value
• **Note** – It is customary to draw the averaged ERH of unit depth in the plot of the unit hydrograph to indicate the type and duration of rainfall creating the unit hydrograph.

• It is assumed that the rainfall excess occurs uniformly over the catchment during the duration D hours of a unit hydrograph
• An ideal duration for a unit hydrograph is one in which small fluctuations in rainfall intensity does not have any significant effect on the runoff
• The duration of the unit hydrograph should not exceed 1/5 to 1/3 of the basin lag
• In general, for catchments larger than 250 sq.km., 6 hour duration is satisfactory.

5.3.4 Unit Hydrograph from a Complex Storm

• When suitable simple isolated storms are not available, data from complex storms of long duration will have to be used to derive the unit hydrograph
• The problem is to decompose a measured composite flood hydrograph into its component DRHs and base flow
• A common unit hydrograph of appropriate duration is assumed to exist
• This is the inverse problem of derivation of the flood hydrograph
• Consider a rainfall excess made up of three consecutive durations of D hours and ER values of R_1, R_2 and R_3.
• After base flow separation of the resulting composite flood hydrograph, a composite DRH is obtained. Let the ordinates of the composite DRH be drawn at a time interval of D hours.
• At various time intervals 1D, 2D, 3D, ……. from the start of the ERH let the ordinates of unit hydrograph be $u_1, u_2, u_3, \ldots$ and the ordinates of the composite DRH be $Q_1, Q_2, Q_3 \ldots$



Fig(3)

Then

$Q_1 = R_1u_1$

$Q_2 = R_1u_2+R_2u_1$

$Q_3 = R_1u_3+R_2u_2+R_3u_1$

$Q_4 = R_1u_4+R_2u_3+R_3u_2$

$Q_5 = R_1u_5+R_2u_4+R_3u_3+$.................. and so on

• The values of $u_1, u_2, u_3, \ldots$ can be determined from the above

• Disadvantage of this method – Errors propagate and increase as computation proceeds

S-Hydrograph

• It is the hydrograph of direct surface discharge that would result from a continuous succession of unit storms producing 1cm(in.)in t_r –hr

• If the time base of the unit hydrograph is Tb hr, it reaches constant outflow (Q_e) at T hr, since 1 cm of net rain on the catchment is being supplied and removed every tr hour and only T/tr unit graphs are necessary to produce an S-curve and develop constant outflow given by,

$$\mathbf{Q_e = (2.78 \cdot A) / t_r}$$

where Q_e = constant outflow (cumec)

t_r = duration of the unit graph (hr)

A = area of the basin (km2 or acres)

• In India, only a small number of streams are gauged (i.e., stream flows due to single and multiple storms, are measured)

• There are many drainage basins (catchments) for which no stream flow records are available and unit hydrographs may be required for such basins

• In such cases, hydrographs may be synthesized directly from other catchments, which are hydrological and meteorologically homogeneous, or indirectly from other catchments through the application of empirical relationship

• Methods for synthesizing hydrographs for ungauged areas have been developed from time to time by Bernard, Clark, McCarthy and Snyder. The best known approach is due to Snyder (1938).

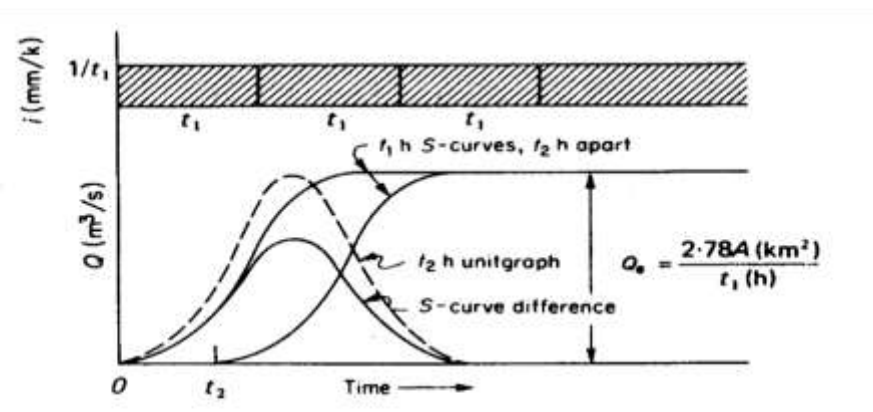


Fig (4) S-Hydrograph

Snyder's method

Snyder (1938) was the to develop a synthetic UH based on a study of watersheds in the Appalachian Highlands. In basins ranging from 10 – 10,000 mi.^{2}

Snyder relations are

$$\mathbf{t_p = Ct\ (LL_C)0.3}$$

where

t_p= basin lag (hr)

L= length of the main stream from the outlet to the divide (mi)

L_c = length along the main stream to a point nearest the watershed centroid (mi)

C_t = Coefficient usually ranging from 1.8 to 2.2

$$\mathbf{Q_p = 640\ C_pA/t_p}$$

where Qp = peak discharge of the UH (cfs)

A = Drainage area (mi^2)

C_p = storage coefficient ranging from 0.4 to 0.8, where larger values of cp are associated with smaller values of C_t

$$\mathbf{T_b = 3+t_p/8}$$

where T_b is the time base of hydrograph

Note: For small watershed the above eq. should be replaced by multiplying tp by the value varies from 3-5

• The above 3 equations define points for a UH produced by an excess rainfall of duration

$$\mathbf{D= t_p/5.5}$$

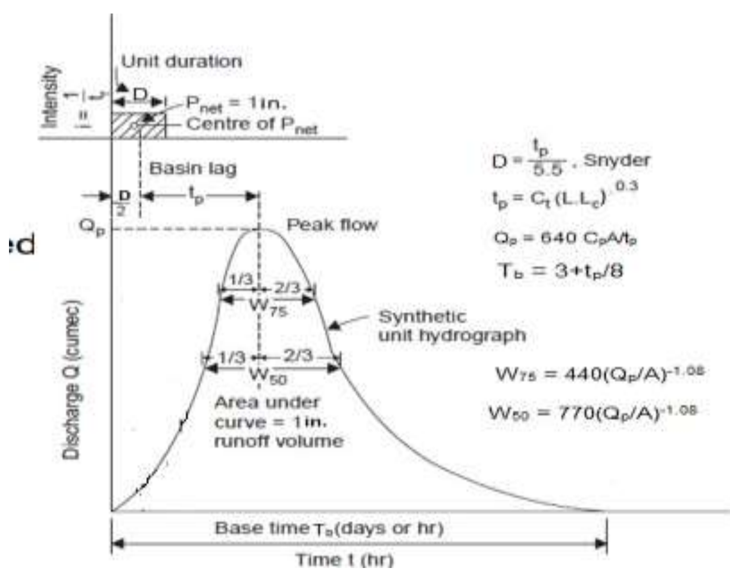


Fig (5) Snyder's hydrograph parameter

Instantaneous Unit Hydrograph (IUH).

• The instantaneous unit hydrograph is defined as a unit hydrograph produced by an effective rainfall of 1 mm and having an infinitesimal reference duration (in other words the duration tends towards zero).

• IUH is the direct runoff hydrograph resulted from an impulse function rainfall i.e., one unit of effective rainfall at a time instance.

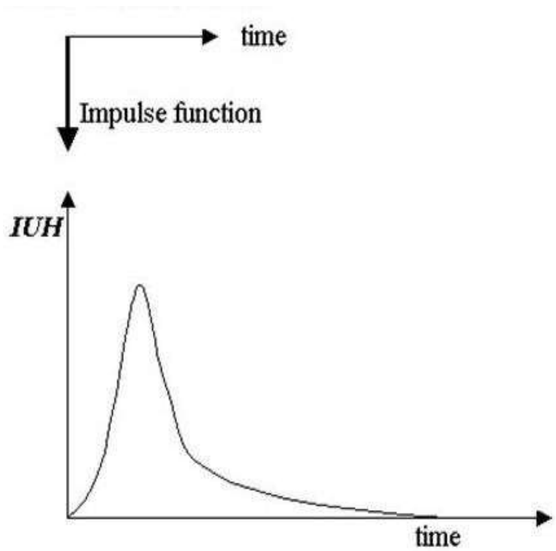


Fig (6) IUH

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Chapter 4

RESERVOIR

SELECTION OF RESERVOIR CAPACITY

The determination of the required capacity of a storage reservoir is usually called an 'opera- tion study' using a long-synthetic record. An operation study may be performed with annual, monthly, or daily time intervals; monthly data are most commonly used.

When the analysis involves lengthy synthetic data, a computer is used and a sequent-peak algorithm is commonly used. Values of the cumulative sum of inflow minus withdrawals taking into account the precipitation, evaporation, seepage, water rights of the downstream users, etc., are calculated, (Fig. 16.15). The first peak and the next following peak, which is greater than the first peak, i.e., the sequent, peak, are identified.

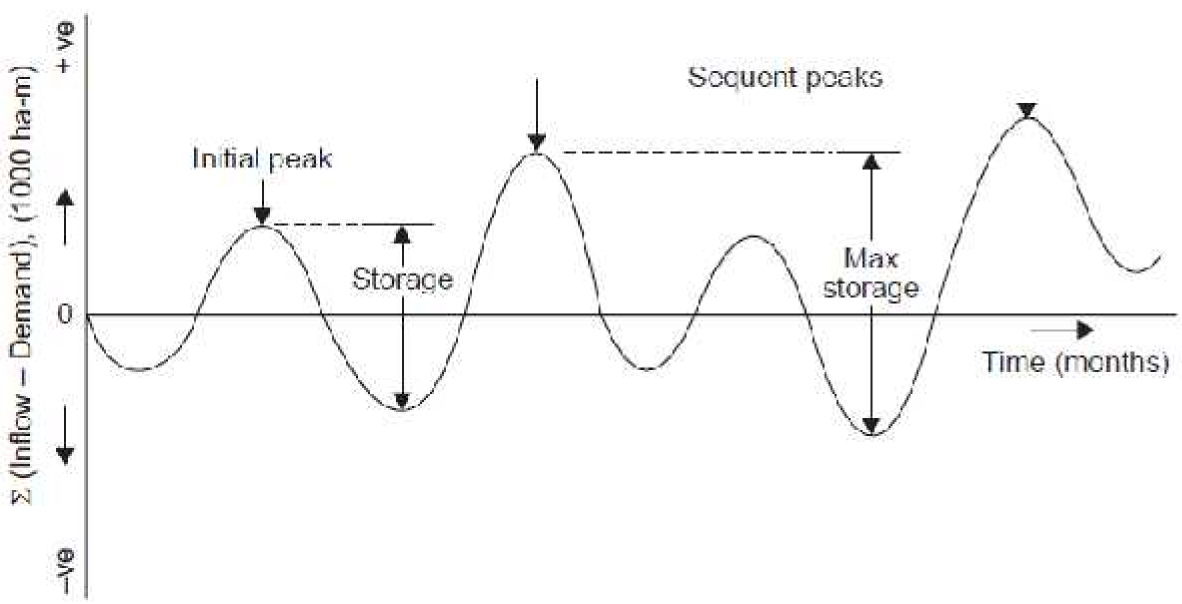


Fig. 16.15 Sequent-peak algorithm

The maximum difference between this sequent peak and the lowest trough during the period under study is taken as the required storage capacity of the reservoir.

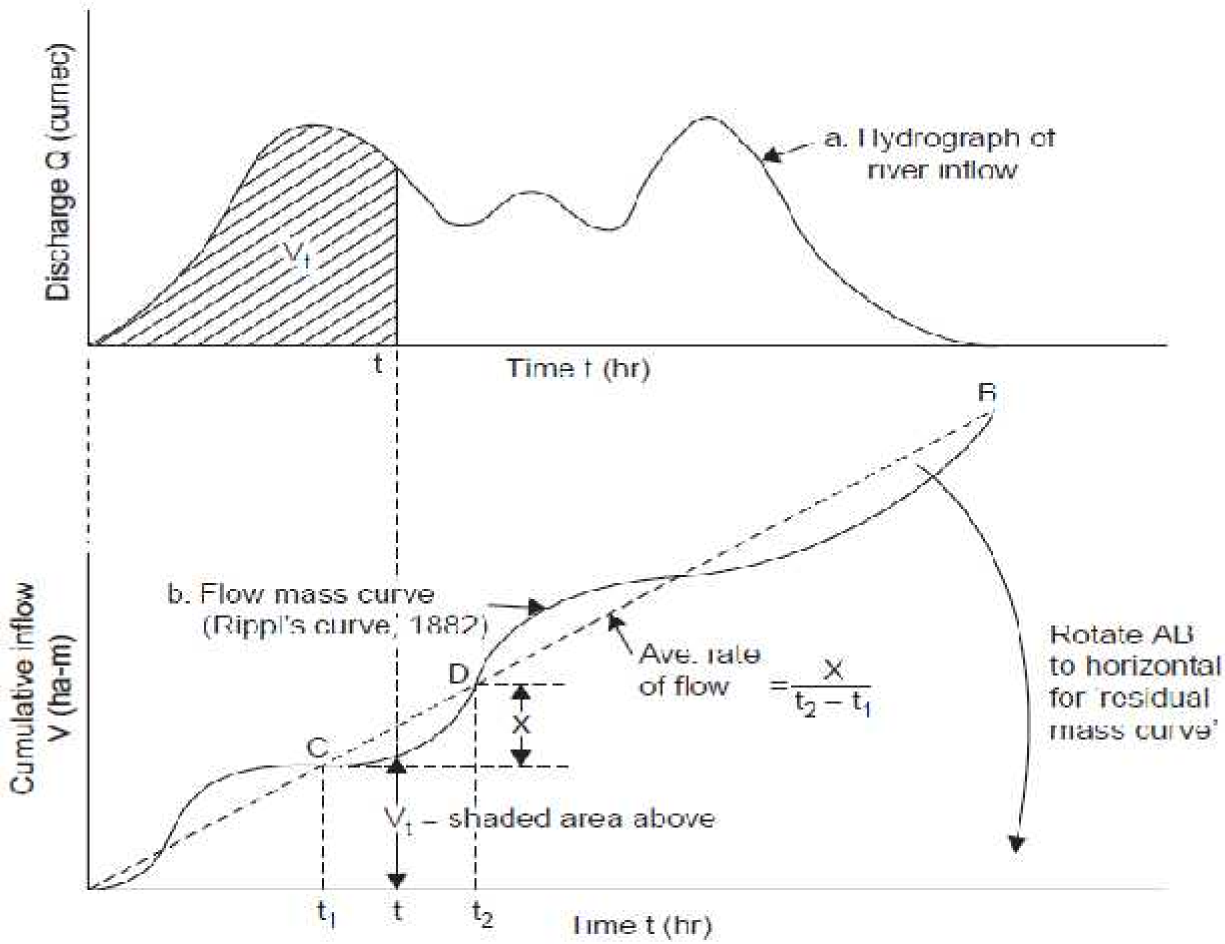
RESERVOIR MASS CURVE

A mass curve (or Rippl diagram, 1882) is a cumulative plotting of net reservoir inflow (Fig. 16.14), and is expressed as

$$V(t) = \int_0^t Q(t)\,dt$$

where V(t) = volume of runoff
Q(t) = reservoir inflow

both as functions of time



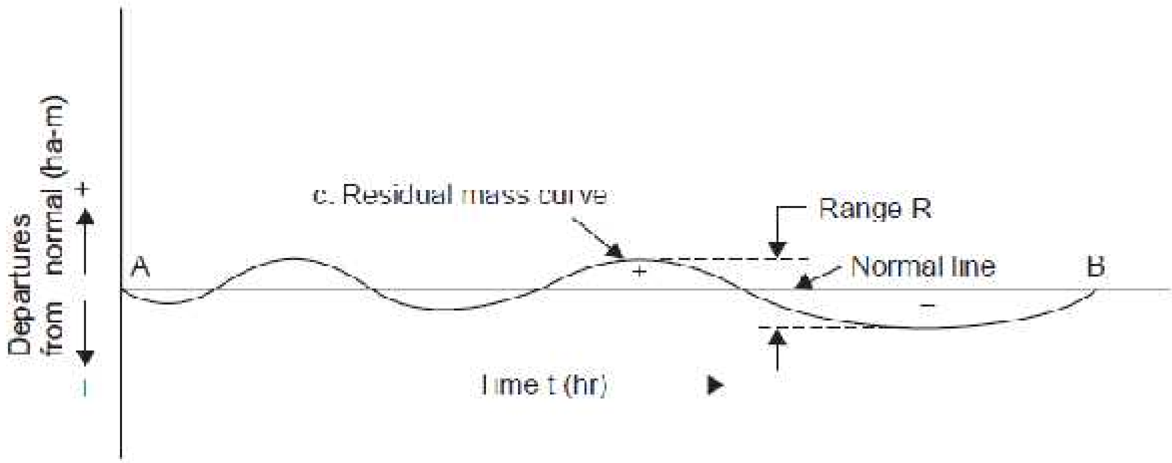


Fig. 16.14 Flow and residual mass curves

The instantaneous rate of flow at any point on the mass curve is given by the slope of the tangent at the point, i.e.

$$Q(t) = \frac{dV(t)}{dt}$$

As already discussed, the mass curve has many useful applications in the design of a storage reservoir, such as determination of reservoir capacity, operations procedure and flood routing.

RESERVOIR MASS CURVE AND STORAGE

During high flows, water flowing in a river has to be stored so that a uniform supply of water can be assured, for water resources utilisation like irrigation, water supply, power generation, etc. during periods of low flows of the river.

A mass diagram is a graphical representation of cumulative inflow into the reservoir versus time which may be monthly or yearly. A mass curve is shown in Fig. 10.1 for a 2-year period. The slope of the mass curve at any point is a measure of the inflow rate at that time. Required rates of draw off from the reservoir are marked by drawing tangents, having slopes equal to the demand rates, at the highest points of the mass curve. The maximum departure between the demand line and the mass curve represents the storage capacity of the reservoir required to meet the demand. A demand line must intersect the mass curve when extended forward, otherwise the reservoir is not going to refill. The vertical distance between the successive tangents represent the water wasted over the spillway. The salient features in the mass curve of flow in Fig. 10.1 are:

a-b: inflow rate exceeds the demand rate of x cumec and reservoir is overflowing

b: inflow rate equals demand rate and the reservoir is just full

b-c: inflow rate is less than the demand rate and the water is drawn from storage

c: inflow rate equals demand rate and S_1 is the draw off from the reservoir (Mm^3)

c-d: inflow rate exceeds demand rate and the reservoir is filling

d: reservoir is full again

d-e: same as a-b e:

similar to b

e-f: similar to b-c

f: inflow rate equals demand rate and S_2 is the draw off from the reservoir

f-g: similar to c-d

To meet the demand rate of x cumec the departure $S_2 > S_1$; hence, the storage capacity of the reservoir is S_2 Mm^3. If the storage capacity of the reservoir, from economic considera- tions, is kept as S_1 Mm^3, the demand rate of x cumec can not be maintained during the time e-f and it can be at a lesser rate of y cumec (y < x).

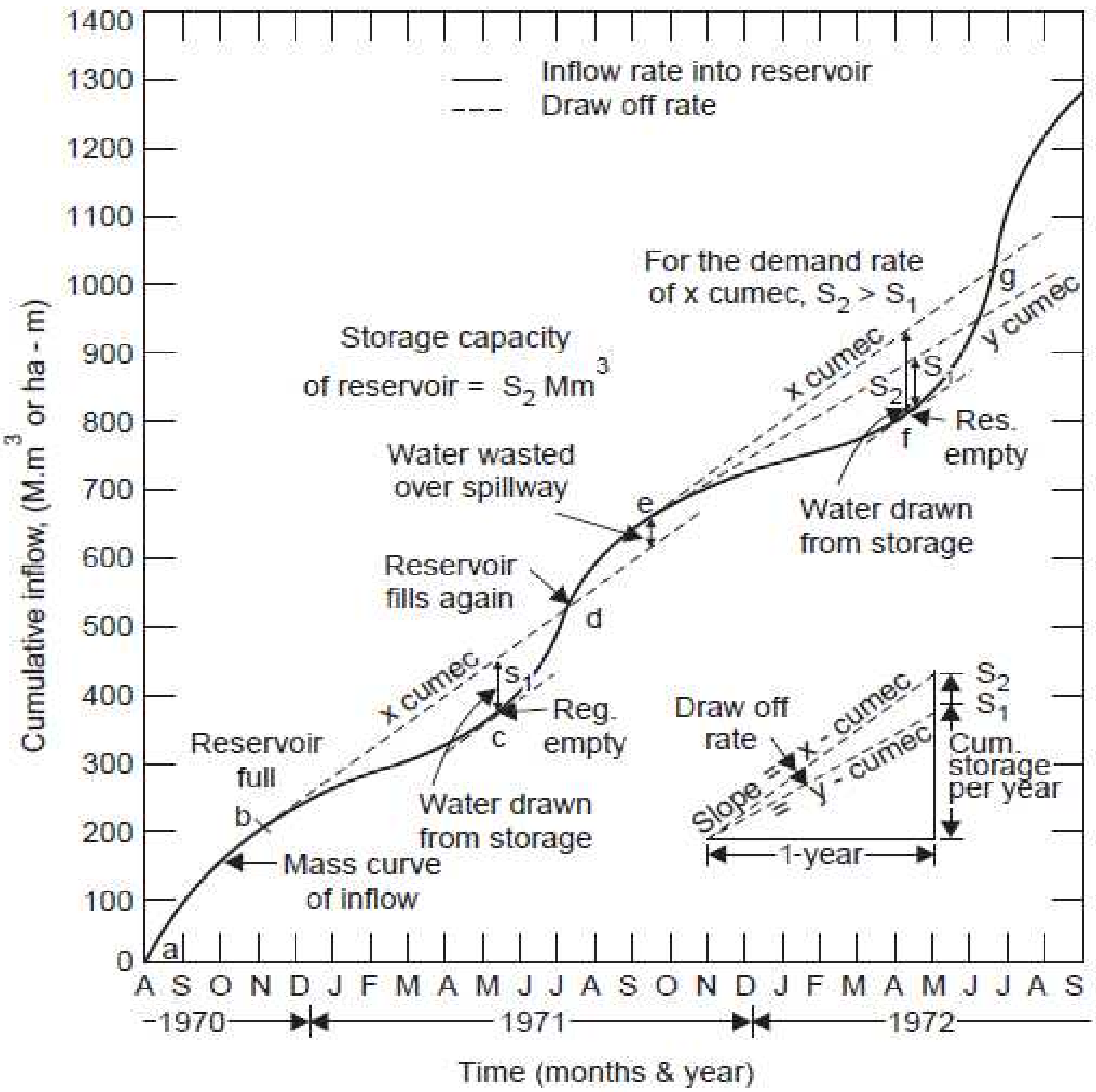


Fig. 10.1 Storage capacity of reservoir from mass curve

The use of mass curve is to determine:

(i) the storage capacity of the reservoir required to meet a particular withdrawal rate. (ii) the possible rate of withdrawal from a reservoir of specified storage capacity.

The observed inflow rates have to be adjusted for the monthly evaporation from the reservoir surface, precipitation, seepage through the dam, inflow from adjacent basins, required releases for downstream users, sediment inflow, etc. while calculating the storage capacity of the reservoir.

The average flow figures for the site of a proposed dam are collected for about 10 years. From this record the flow figures for the driest year are used for drawing the mass flow curve. Graphical analysis is enough for preliminary studies. Final studies are made by tabular computation. If tangents are drawn to the crest and trough of the mass curve such that the departure of the lines represents the specified reservoir capacity, the slope of the tangent at the crest gives the continuous flow that can be maintained with the available storage capacity. From this the greatest continuous power output for the available fall at the site for a given plant efficiency and load factor can be determined.

From the daily flow data a hydrograph or a bar graph is drawn for the maximum flood during the period of 10 years and the spillway capacity to pass this flood with the available storage capacity is determined. Thus, the power and the flood control potentialities of the site are investigated.

The mass curve of water utilisation need not be a straight line. The dashed curve in Fig. 10.2 shows the cumulative requirements of water use in different months as compared with monthly cumulatively inflow. The maximum draft in the reservoir (i.e., maximum departure of the water use and inflow curves) occurs by the end of April. The reservoir again becomes full by the end of September when the two curves intersect.

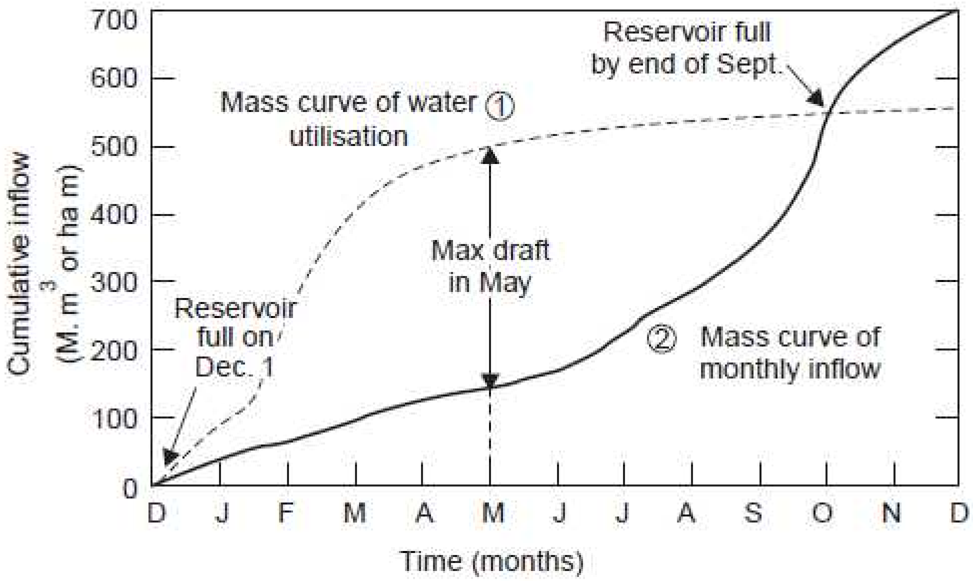


Fig. 10.2 Mass curves of water utilisation and monthly inflow

FLOW DURATION CURVES

Flow duration curves show the percentage of time that certain values of discharge weekly, monthly or yearly were equalled or exceeded in the available number of years of record. The selection of the time interval depends on the purpose of the study. As the time interval in- creases the range of the curve decreases, Fig. 10.4. While daily flow rates of small storms are useful for the pondage studies in a runoff river power development plant, monthly flow rates for a number of years are useful in power development plants from a large storage reservoir. The flow duration curve is actually a river discharge frequency curve and longer the period of record, more accurate is the indication of the long term yield of a stream. A flat curve indicates a river with a few floods with large ground water contribution, while a steep curve indicates frequent floods and dry periods with little ground water contribution.

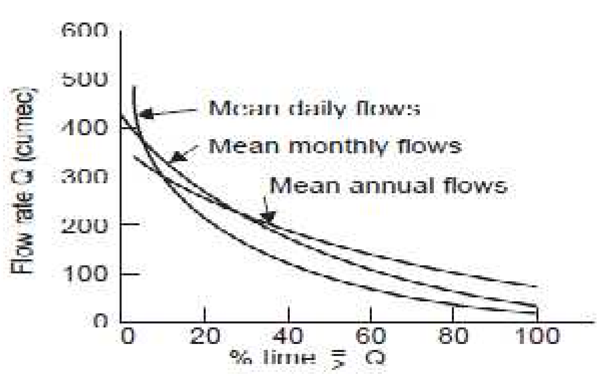


Fig. 10.4 Flow duration curves—effect of observation period

SEDIMENT MOVEMENT AND DEPOSITION

As the silt originates from the water shed, the characteristics of the catchment such its areal extent, soil types, land slopes, vegetal cover and climatic conditions like temperature, nature and intensity of rainfall, have a great significance in the sediment production in the form of sheet erosion, gully erosion and stream, channel erosion. In regions of moderate rain- fall, sheet erosion is the dominant source of total sediment load while in arid and semi-arid regions, gullying and stream-channel erosion furnish the greater part of the load.

Experiments have shown that the erosive power of water, flowing with a velocity V, varies as V^2 *while the transporting ability of water varies as* V^6*. Sediment moves in the stream as suspended load (fine particles) in the flowing water, and as bed load (large parti- cles), which slides or rolls along the channel bottom. Sometimes, the particles (small particles of sand and gravel) move by bouncing along the bed, which is termed as 'saltation', which is a transitional stage between bed and suspended load. The material, which moves as bed load at one section may be in suspension at another section.*

The suspended sediment load of streams is measured by sampling the water, filtering to remove the sediment, drying and weighing the filtered material.

The samplers may be of 'depth-integrating type' or 'point samplers'. Point samplers are used only where it is not possible to use the depth integrating type because of great depth of high velocity, or for studies of sediment distribution in streams. The sample is usually col- lected in 'pint bottle' held in a sample of stream-lined body so as not to disturb the flow while collecting a representative sample.

The relation between the suspended-sediment transport Q_s *and stream flow Q is given*

$$Q_s = KQ^n$$
$$\log Q_s = \log K + n \log Q$$

and is often represented by a logarithmic plot of Q_s *vs. Q (Fig. 11.1);* $Q_s = K$ *when Q = 1, and n is the slope of the straight line plot and 2 to 3.*

The sediment rating curve from a continuous record of stream flow provides a rough estimate of sediment inflow to reservoirs and the total sediment transport may be estimated by adding 10-20% to the suspended sediment transport to allow for the bed load contribution.

When the sediment-laden water reaches a reservoir, the velocity and turbulence are greatly reduced. The dense fluid-solid mixture along the bottom of the reservoir moves slowly in the form of a density current or stratified flows, i.e., a diffused colloidal suspension having a density slightly different from that of the main body of reservoir water, due to dissolved minerals and temperature, and hence does not mix readily with the reservoir water (Fig. 11.2). Smaller particles may be deposited near the base of the dam. Some of the density currents and settled sediments near the base of the dam can possibly be flushed out by operating the sluice gates. The modern multipurpose reservoirs are operated at various water levels, which are

significant in the deposition and movement of silt in the reservoir.

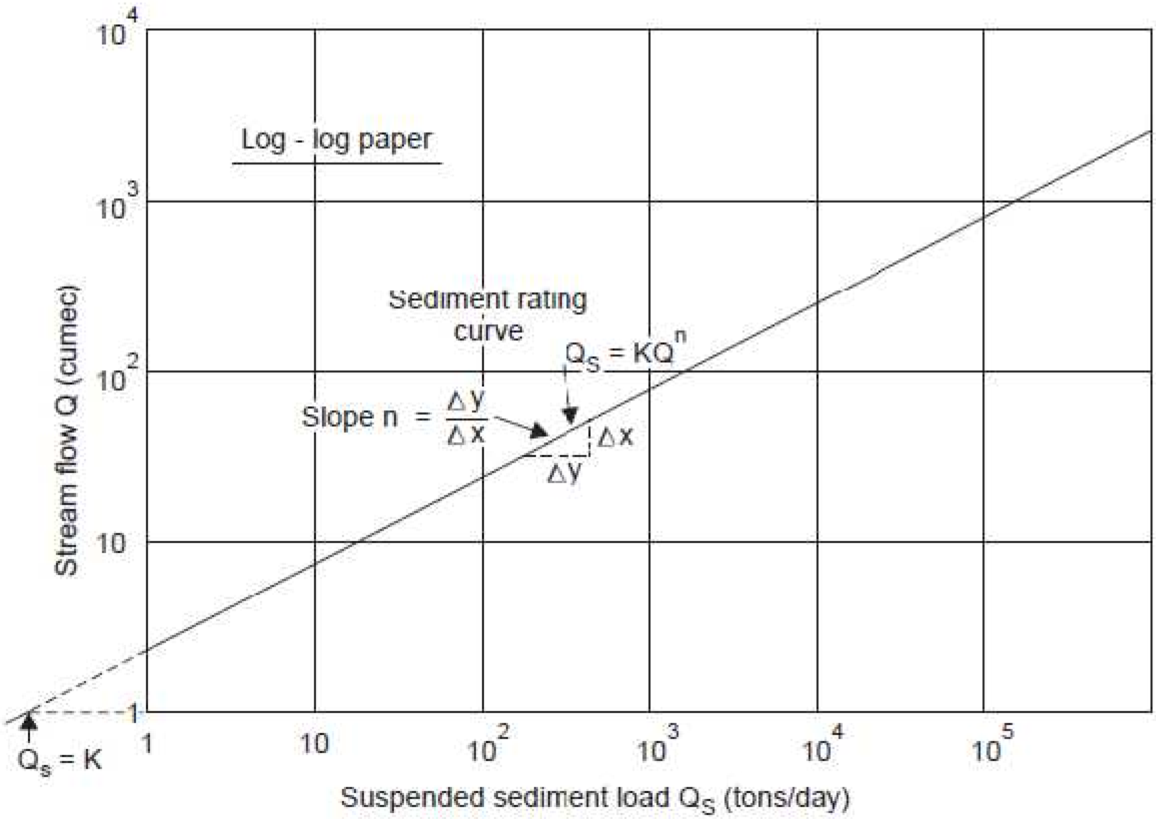


Fig. 11.1 Sediment rating curve

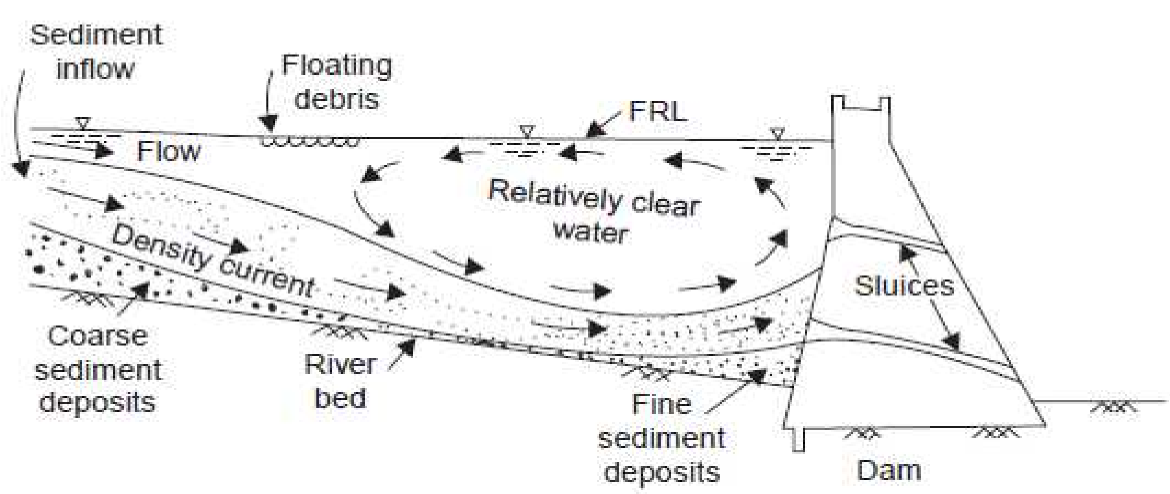


Fig. 11.2 Sediment accumulation in a reservoir

The total amount of sediment that passes any section of a stream is referred to as the sediment yield or sediment production. The mean annual sediment production rates generally range from 250-2000 tons/km^2 or 2.5-18 ha-m/100 km^2 and the Indian reservoirs are losing a storage capacity of 0.5-1% annually.

REDUCTION IN RESERVOIR CAPACITY

The useful life of a reservoir gets reduced due to sediment deposition causing a decrease in its storage capacity.The factors affecting the pattern of sediment deposition in reservoirs are:

(i) sediment load (i.e., sediment inflow rate) (ii)

sediment size (i.e., gradation of silt)

(iii) compaction of sediment (iv) river inflow pattern

(v) river valley slope (vi) shape of reservoir

(vii) capacity of reservoir (its size and storage period)

(viii) vegetal growth at the head

(ix) outlets in the dam (their types, location and size) (x)

reservoir operation

(xi) upstream reservoirs, if any.

It has been found by experience that a low sediment inflow rate, large fraction of fine particles, steep slope, no vegetation at head of reservoir, low flow detention time in the reservoir (by operation of outlets of suitable size at different levels), possibly series of upper tanks or reservoir upstream (where deposition occurs) do not favour sediment deposition and compaction. The silt carried in the rainy season may be excluded from the reservoir by means of scouring sluices slightly above the deep river-bed, which discharge the heavily silt-laden water at high velocity. The percent of the inflowing sediment, which is retained in a reservoir is called the trap efficiency and it is a function of the ratio of reservoir capacity to total annual sediment inflow, since a small reservoir on a large stream passes most of its inflow quickly (giving no time for the silt to settle) while a large reservoir allows more detention time for the suspended silt to settle. The relation between trap efficiency of reservoir vs. capacity-inflow ratio is shown in Fig. 11.3 (Brune, 1953), on the basis of data from surveys of existing reservoirs. The rate at which the capacity of a reservoir is reduced by sediment deposition depends on

(i) the rate of sediment inflow, i.e., sediment load.

(ii) the percentage of the sediment inflow trapped in the reservoir, i.e., trap efficiency. (iii)

the density of the deposited sediment.

RESERVOIR SEDIMENTATION CONTROL

Sediment deposition in reservoirs can not be actually prevented but it can be retarded by adopting some of the following measures:

(i) Reservoir sites, which are prolific sources of sediment should be avoided.

(ii) By adopting soil-conservation measures in the catchment area, as the silt originates in the watershed. See art 8.7 in Chapter 8.

(iii) Agronomic soil conservation practices like cover cropping, strip cropping, contour farming, suitable crop rotations, application of green manure (mulching), proper control over graze lands, terracing and benching on steep hill slopes, etc. retard overland flow, increase infiltration and reduce erosion.

(iv) Contour trenching and afforestation on hill slopes, contour bunding gully plugging by check dams, and stream bank stabilisation by the use of spurs, rivetments, vegetation, etc. are some of the engineering measures of soil conservation.

(v) Vegetal cover on the land reduces the impact force of rain drops and minimises erosion.

(vi) Sluice gates provided in the dam at various levels and reservoir operation, permit the discharge of fine sediments without giving them time to settle to the bottom.

(vii) Sediment deposits in tanks and small reservoirs may be removed by excavation, dredging, draining and flushing either by mechanical or hydraulic methods and sometimes may have some sales value.

Floods Management & Hydrologic Analysis

SIZE OF FLOODS

A flood is an unusual high stage of a river due to runoff from rainfall and/or melting of snow in quantities too great to be confined in the normal water surface elevations of the river or stream, as the result of unusual meteorological combination.

The maximum flood that any structure can safely pass is called the 'design flood' and is selected after consideration of economic and hydrologic factors. The design flood is related to the project feature; for example, the spillway design flood may be much higher than the flood control reservoir design flood or the design flood adopted for the temporary coffer dams. A design flood may be arrived by considering the cost of constructing the structure to provide flood control and the flood control benefits arising directly by prevention of damage to structures downstream, disruption communication, loss of life and property, damage to crops and under- utilisation of land and indirectly, the money saved under insurance and workmen's compensation laws, higher yields from intensive cultivation of protected lands and elimination of losses arising from interruption of business, reduction in diseases resulting from inundation of flood waters. The direct benefits are called tangible benefits and the indirect benefits are called intangible benefits. The design flood is usually selected after making a cost-benefit analysis and exercising engineering judgement.

When the structure is designed for a flood less than the maximum probable, there exists a certain amount of flood risk to the structure, nor is it economical to design for 100% flood protection. Protection against the highest rare floods is uneconomical because of the large investment and infrequent flood occurrence.

In the design flood estimates, reference is usually made to three classes:

(a) Standard Project Flood (SPF). This is the estimate of the flood likely to occur from the most severe combination of the meteorological and hydrological conditions, which are rea- sonably characteristic of the drainage basin being considered, but excluding extremely rare combination.

(b) Maximum Probable Flood (MPF). This differs from the SPF in that it includes the extremely rare and catastrophic floods and is usually confined to spillway design of very high dams. The SPF is usually around 80% of the MPF for the basin.

(c) Probable Maximum Precipitation (PMP). From the observations of air moisture from the maximum due-point and temperature recorded and air-inflow (from the wind speed and barometric pressure recorded), the moisture inflow index in the storm is determined. The best known upward adjustment to be applied to the historical and hypothetical major storms is the

maximisation with respect to moisture charge. The adjusted storm rainfall is assumed to bear the same ratio to the observed storm rainfall, as the maximum moisture charge over the basin to the moisture charge of the observed storm. From the critical combinations of storms, and moisture adjustment the PMP is derived which, after minimising losses, when applied on the design unit hydrograph for the basin, will produce the MPF. Occasionally when enough storm data for the given basin is not available, PMP can be estimated by adopting a severe storm over neighbouring catchment (which is meteorologically homogeneous) and transposing it to the catchment under consideration.

(d) Design Flood—It is the flood adopted for the design of hydraulic structures like spillways, bridge openings, flood banks, etc. It may be the MPF or SPF or a flood of any desired recurrence interval depending upon the degree of flood protection to be offered and cost eco- nomics of construction of structures to the desired flood stage; the design flood is usually se- lected after making a cost-benefit analysis, i.e., the ratio of benefit to cost may be desired to be the maximum.

ESTIMATION OF PEAK FLOOD

The maximum flood discharge (peak flood) in a river may be determined by the following methods:

- *(i) Physical indications of past floods—flood marks and local enquiry*
- *(ii) Empirical formulae and curves*
- *(iii) Concentration time method*
- *(iv) Overland flow hydrograph*
- *(v) Rational method*
- *(vi) Unit hydrograph*
- *(vii) Flood frequency studies*

The above methods are discussed below:

(i) Observations at nearby structure. By noting the flood marks (and by local enquiry), depths, affluxes (heading up of water near bridge openings, or similar obstructions to flow) and other items actually at an existing bridge, on anecut (weir) in the vicinity, the maximum flood discharge may be estimated. The flood marks are connected by levelling, the profile is plotted and HFL marked on it, and the cross sectional area is determined. The surface fall at HFL is calculated from the difference in HFL at known distance apart. It may be checked with the bed slope; $Q = CA^{3/4}$

the coefficient C = 11–14, where the aar is 60–120 cm

= 14–19 in Madhya Pradesh

= 32 in western Ghats

up to 35, maximum value

2. Ryves formula derived from a study of rivers in south India

$$A = CA^{2/3}$$

Coefficient C = 6.8 within 80 km of coast

= 8.3 for areas between 80 and 2400 km from the coast

= 10.0 for limited area near the hills up to 40,

actual observed values

3. *Inglis formula for fan-shaped catchments of Bombay state (Maharashtra)*

$$Q = \frac{124\,A}{\sqrt{A \quad 10.4}}$$

4. *Myers formula* $Q = 175\ A^{0.5}$

5. *Ali Nawab Jang Bahadur formula for the old Hyderabad state*

$$Q = CA^{(0.993 - 1/14 \log A)}$$.

the coefficient C varies from 48 to 60

Maximum value of C = 85

6. *Fuller's formula (1914)*

$$Q = CA^{0.8}\,(1 + 0.8 \log T_T)(1 + 2.67\ A^{-0.3})$$. *constants derived from the basins in USA 10 years data is required for sufficient reliability. The coefficient C varies from 0.026 to 2.77; T = recurrence interval in years. Fuller was the first to suggest that frequency should be considered as a factor in estimating floods.*

7. *Greager's formula for USA*

$$Q = C(0.386\ A)^{0.894(0.386\ A)}\ {}_{0.048}$$

8. *Burkli Ziegler formula for USA*

$$Q = 412\ A^{3/4}$$

In all the above formula, Q is the peak flood in cumec and A is the area of the drainage basin in km^2.

(iii) Envelope Curves. Areas having similar topographical features and climatic conditions are grouped together. All available data regarding discharges and flood formulae are compiled along with their respective catchment areas. Peak flood discharges are then plotted against the drainage areas and a curve is drawn to cover or envelope the highest plotted points. Envelope curves are generally used for comparison only and the design floods got by other methods, should be higher than those obtained from envelope curves. For Indian rivers, envel- oping curves from observed floods have been developed by Kanwar Sain and Karpov, Fig. 8.1 (a).

(iv) Concentration Time Method. The concentration time method of estimating the peak discharge consists of two steps:

(i) Determination of the concentration time, etc.

(ii) Selection of the period of maximum net rainfall for the concentration time duration. This method can be used for design storms or in conjunction with intensity-duration- frequency curves.

(v) Rational Method. The rational method is based on the application of the formula

$$Q = CiA \qquad \text{...(8.9)}$$

where C is a coefficient depending on the runoff qualities of the catchment called the runoff coefficient (0.2 to 0.8). The intensity of rainfall i is equal to the design intensity or critical intensity of rainfall i_c corresponding to the time of concentration t_c for the catchment for a given recurrence interval T; the design intensity of rainfall i (= i_c) can be found from the intensity-duration-frequency curves, for the catchment corresponding to t_c and T. If the intensity-duration-frequency curves, are not available for the catchment and a maximum precipitation of P cm occurs during a storm period of t_R hours, then the design intensity i (= i_c) can be obtained from the equation

(vi) The Unit Hydrograph Method. For small and medium size basins ($A < 5000$ km^2, i.e., when a single unit hydrograph could be applied to the entire basin) in developing design flood hydrographs by applying the unit hydrograph for the basin, the design storm estimates are made by the following methods.

(i) Selection of major storms

(ii) Maximization of selected storms

(iii) Plotting the depth-area-duration curves and their analysis

(iv) Moisture adjustment

(v) Storm transposition to a critical position

(vi) Envelopment of the transposed adjusted storms

(vii) Use of minimum infiltration indices

In the depth-area-duration analysis of a particular storm, the maximum average depths of rainfall over various sizes of area during certain periods of storm (hr or days), say cm over 1000 km^2 in 1 day, 2 days or 3 days from the isohyetal maps contructed (see Fig. 2.15). Such values determined for all the transposable storms provide the basic data to estimate the PMP over the basin.

FLOOD FREQUENCY STUDIES

When stream flow peaks are arranged in the descending order of magnitude they constitute a statistical array whose distribution can be expressed in terms of frequency of occurrence. There are two methods of compiling flood peak data—the annual floods and the partial duration series. In the annual floods, only the highest flood in each year is used thus ignoring the next highest in any year, which sometimes may exceed many of the annual maximum. In the par- tial duration series, all floods above a selected minimum are taken for analysis, regardless of the time-interval, so that in some years there may be a number of floods above the basic stage, while in some other years there may not any such flood at all. The disadvantage of the partial duration series is that the data do not furnish a proper frequency (true distribution) series and so a reasonable statistical analysis cannot be made. But all the larger floods are used in this analysis, which is an advantage while in annual flood series some big floods are omitted because they were not the highest floods in any year considered. Usually the basic stage is as- sumed sufficiently low so that as many peaks (4 or 5) as

possible each year are above this stage. The two series give very nearly the same recurrence interval for the larger floods, but the partial series indicates higher floods for shorter recurrence intervals. For information about floods of fairly frequent occurrence, as is required during the construction period of a large dam (say, 4-5 years), the partial series are the best, while for the spillway design flood the annual series are preferable, since the flood should not be exceeded in the dam's life time, say 100 years.

Annual Flood Series The return period or recurrence interval (T) is the average number of years during which a flood of given magnitude will be equalled or exceeded once and is computed by one of the following methods.

California method (1923):

$$T = \frac{n}{m}$$

Weilbul method (1939):

$$T = \frac{n \quad 1}{m} \qquad \text{...(8.11 b)}$$

where n = number of events, i.e., years of record

m = order or rank of the event (flood item) when the flood magnitudes (items) are ar- ranged in the descending order ($m = 1$ for the highest flood, $m = n$ for the lowest flood)

T = recurrence interval ($T = n$-yr for the highest flood, $T = 1$ yr for the lowest flood, by California method)

The probability of occurrence of a flood (having a recurrence interval T-yr) in any year, i.e., the probability of exceedance, is

$$P = \frac{1}{T} \qquad \text{...(8.12)}$$

and the probability that it will not occur in a given year, i.e., the probability of non-exceedance (P), is

$$P \quad = 1 - P \qquad \text{...(8.12 b)}$$

One interesting example of the application of statistics to a hydrologic problem (i.e., stochastic hydrology), is Gumbel's theory of extreme values. The probability of an event of magnitude x not being equalled or exceeded (the probability of non-occurrence, P), based on the argument that the distribution of floods is unlimited (i.e., for large values of n, say $n > 50$),

METHODS OF FLOOD CONTROL

A flood is an unusual high stage of a river overflowing its banks and inundating the marginal lands. This is due to severe storm of unusual meteorological combination, sometimes combined with melting of accumulated snow on the catchment. This may also be due to shifting of the course of the river, earthquake causing bank erosion, or blocking of river, or breaching of the river flood banks. Floods have swept vast regions in India, particularly in the basins of rivers Kosi, Brahmaputra, Godavari, Narmada and Tapti. Floods cause much loss of life and property, disruption of communication, damage to crops, famine, epidemic diseases and other indirect losses.

Design magnitudes of floods are needed for the design of spillways, reservoirs, bridge openings, drainage of cities and air ports, and construction of flood walls and levees (flood banks). The maximum flood that any structure can safely pass is called the 'design flood'.

The damages due to the devastating floods can be minimised by the following flood control measures, singly or in combination.

(i) by confining the flow between high banks by constructing levees (flood banks) , dykes, or flood walls.

(ii) by channel improvement by cutting, straightening or deepening and following river training works.

(iii) by diversion of a portion of the flood through bypasses or flood ways. In some cases a fuse plug levee is provided. It is a low section of levee, which when once over topped, will wash out rapidly and develop full discharge capacity into the flood-way. In other locations, a concrete sill, weir or spillway controlled by stop logs or needles may be provided so that the overflow occurs at a definite river stage. Sometimes dynamiting a section of levee is resorted to bypass the flood.

(iv) by providing a temporary storage of the peak floods by constructing upstream reser- voirs and retarding basins (detention basins).

(v) by adopting soil conservation measures (land management) in the catchment area. (vi) by temporary and permanent evacuation of the flood plain, and flood plain zoning by enacting legislation.

(vii) by flood proofing of specific properties by constructing a ring levee or flood wall around the property.

(viii) by setting up flood forecasting—short term, long term, rhythm signals and radar, and warning centres at vulnerable areas.

Flood Control by Reservoirs

The purpose of a flood control reservoir is to temporarily store a portion of the flood so that the flood peaks are flattened out. The reservoir may be ideally situated immediately upstream of the area to be potected and the water discharged in the channel downstream at its safe capac- ity (known from its stage-discharge curve), i.e., the peak has been reduced by AB, Fig. 8.6. All the inflow into the reservoir in excess of the safe channel capacity is stored until the inflow drops below the channel capacity and the stored water is released to recover the storage capacity for the next flood.

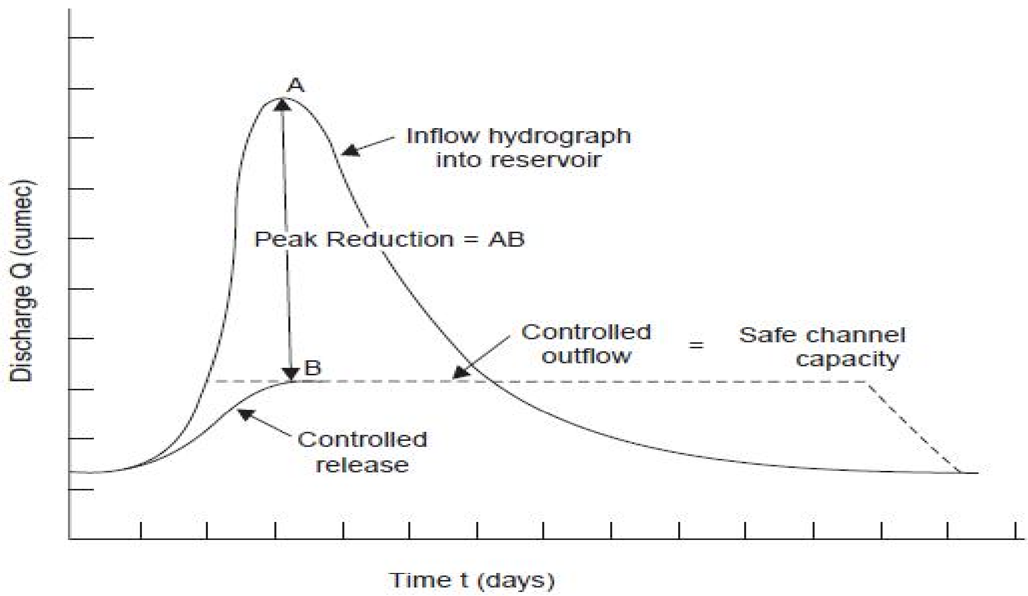


Fig. 8.6 Flood control by reservoirs

If there is some distance between the reservoir and the protected area but no local inflow between these points, the reservoir operation is similar to the above but the peak will further be reduced due to storage in the reach downstream from the reservoir. If there is a substantial local inflow between the dam and the control point, the reservoir must be operated to produce a minimum peak at the protected area rather than at the dam site; otherwise the release from a reservoir may unfortunately synchronise at some point downstream with flood flows from a tributary. Timely and reliable weather forecasts and prompt information about precipitation upstream and downstream of the reservoir and the means of translation of this information into necessary flood hydrographs will all help in effective reservoir operation. Construction of reservoirs for flood control only is rarely economical and other benefits like irrigation, hydel power, also have to be taken into consideration. It is the modern practice to construct multipurpose reservoirs, where a space is allocated exclusively for flood control, usually above the spillway crest level and is made available when required by closing the spillway crest gates.

The effectiveness of the reservoir in reducing peak flows, increases as its storage capacity increases. The maximum capacity required is the difference in volume between the safe release from the reservoir and the maximum inflow. Since the hydrograph is wider at low flows, more water must be stored to reduce the peak by a given amount. As the peak reduction is increased, more marginal area will be protected from the floods. The benefits accrued by a unit peak reduction are usually less. Thus, the size of the reservoir has to be determined by weighing the cost involved in reducing the peak with the benefits accrued. A

single reservoir across the main river may not give the required protection to all towns and cities widely lo- cated and reservoirs constructed across tributaries are effective in flood protection. In general, at least one-third of the total drainage area should come under one reservoir for effective flood control.

Retarding Basins

The release from a storage reservoir is controlled by gates and valves and regulated by the project engineer. A retarding basin is provided with outlets like a large spillway and sluices with no control gates. The sluice discharges like an orifice, i.e., $Q = C_d A \sqrt{2gH}$*, and there is a greater throttling of flow when the reservoir is nearly full than would a spillway discharging like a weir, i.e.,* $Q = CLh^{3/2}$ *(Fig. 8.7). However, a spillway is necessary for emergency in case of the flood exceeding the design maximum.*

The discharge capacity of a retarding basin when full should equal the safe discharging capacity of the channel downstream. The storage capacity of the basin should be equal to the volume of the design flood minus the volume of water released during the flood. A retarding basin is used only for the purpose of flood control. After the peak of flood has passed, the inflow will gradually become equal to the outflow. One of the limitations of the retarding basins is that the discharge from the basin may synchronise with the flood flow of a tributary down- stream and as such they are constructed on comparatively small stream while storage reser- voirs are provided across big rivers (since the release can be regulated).

For any pool elevation of the reservoir, the storage and discharge can be calculated. For a known inflow hydrograph, the corresponding outflow hydrograph can be determined by any

method of flood routing

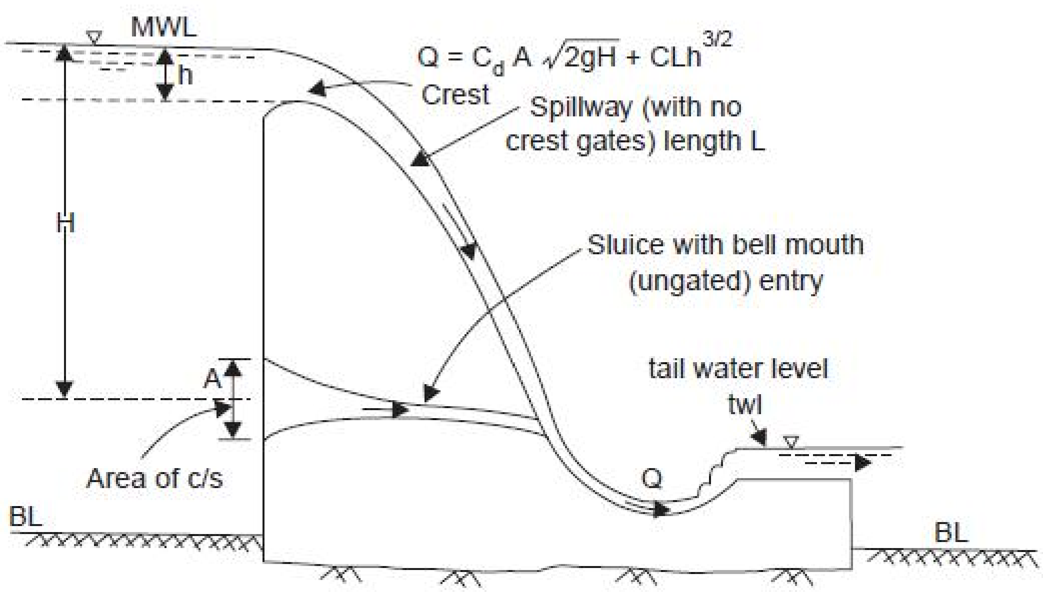


Fig. 8.7 Retarding basin

Construction of Levees

The construction of levees (flood banks or dykes) is extensively followed in India, since it is an economical, direct and immediate method. The design and construction of levees are similar to those of an earth dam. The levees are constructed beyond the meander belt of a river, Fig. 8.8 (a) and they tame a river not to change its course. As far as possible, there should be very few curves in their alignment. They require constant watch and after the floods recede, repairs and restoration of levees should be resorted to.

The spacing and height of levees are determined by a series of trials. A height is assumed and the discharge through the proper channel is computed for the assumed high water flow, which is the level of the top of bank less the free board. This flow subtracted from the estimated probable maximum flood discharge gives the discharge to be passed over the flood ways between the proper channel and the levees (Figs. 8.8 (a) and (b), 8.9 and 8.10). Area of the flood ways is then obtained by dividing it by the velocity of flow. The spacing of levees thus obtained, should give a minimum value for the cost of levees and the value of the submerged land in the flood way. The effects of levees on flood flow are:

(i) increase in the rate of flood flow

(ii) increase in the flood water elevation

(iii) increase in the carrying capacity of the channel

(iv) increase in the scouring action

(v) decrease of surface slope of stream above the leveed section

Channel Improvement

Channel improvement increases the discharging capacity of the stream thereby decreasing the height and duration of the flood. Flood carrying capacity can be increased either by in- creasing the cross-sectional area or by increasing the velocity along the river. Enlarging the section is attempted only for narrow and shallow channels with small watersheds, the limit of such enlargement in width being 30-40 m. Deepening is preferred to widening since the hy- draulic mean radius increases more with depth (for the same increase in the sectional area) thus increasing the velocity.

The channel velocity (given by Manning's or Chezy's formulae) is affected by hydraulic mean radius, slope of river bed and roughness of the bed and sides. Roughness can be reduced by

(i) removing sand bars.

(ii) prevention of cropping on river beds near banks.

(iii) removal of fallen trees and other snags.

(iv) elimination of sharp bends of meandars by providing cutoffs (Fig. 8.11).

In a stream, deepening results in the loss of slope as its outlet can not usually be low-

ered. Deepening can be resorted to only when cutoffs are provided, when the slope of the channel is increased due to the reduction in the length of flow. Thus, a cutoff helps

(i) in increase the velocity by increasing the slope,

(ii) to shorten the path of flow by elimination of meandars, and consequently

(iii) to shorten the levees necessary to confine flood waters.

Elimination of meanders by providing straight cutoffs has been done on the river Mississippi in USA.

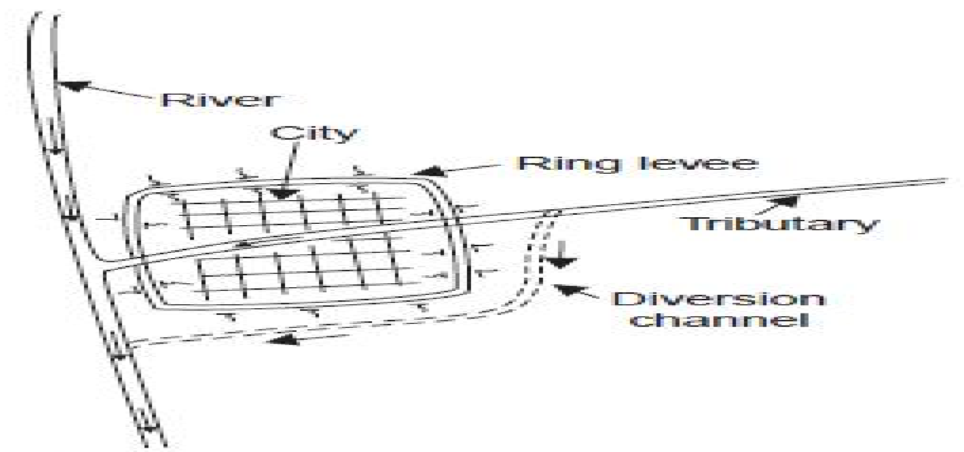
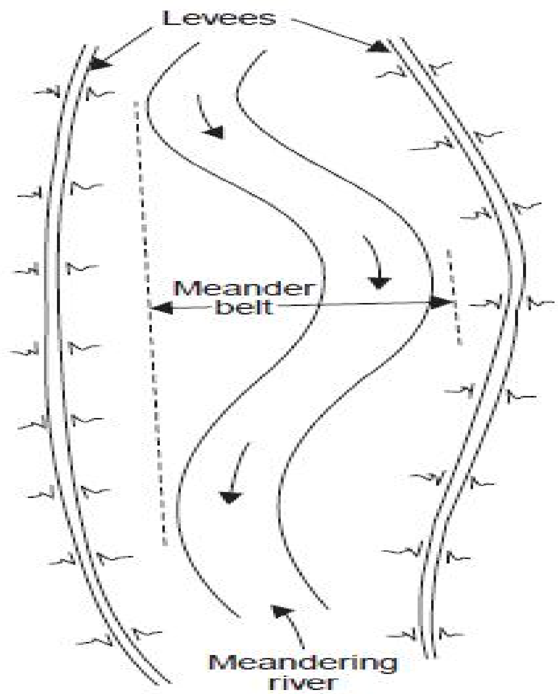
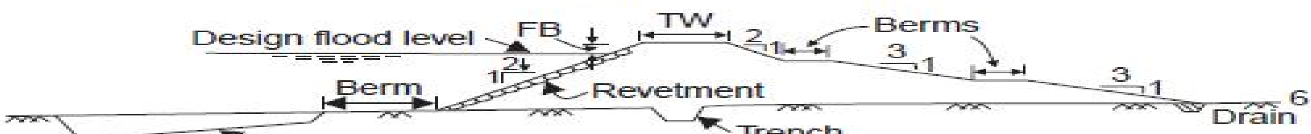


Fig. 8.9 Ring levee to protect a city



(a) Levees along a meandering river



FB - Free board, TW - Top width

(b) Typical levee cross section

Fig. 8.8 Flood control by levees

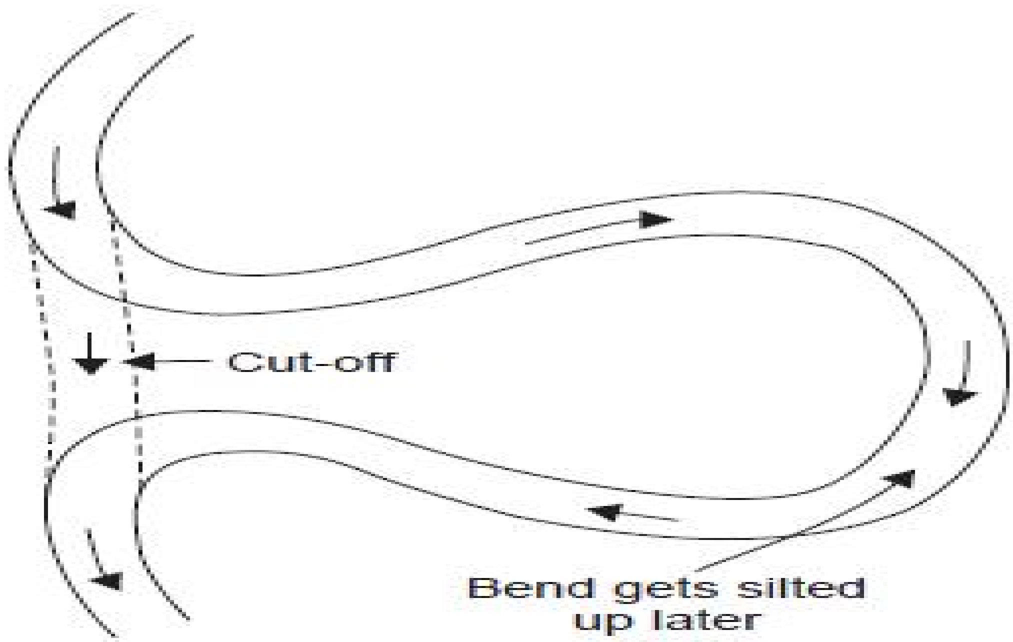


Fig. 8.11 Cut-off in a meandering river

RESERVOIR ROUTING

Flood routing is the process of determining the reservoir stage, storage volume of the outflow hydrograph corresponding to a known hydrograph of inflow into the reservoir; this is called reservoir routing. For this, the capacity curve of the reservoir, i.e., 'storage vs pool elevation', and 'outflow rate vs. pool elevation', curves are required. Storage volumes for different pool elevations are determined by planimetering the contour map of the reservoir site. For exam- ple, the volume of water stored (V) between two successive contours having areas A_1 and A_2 (planimetered) and the contour interval d, is given by

Cone formula, $$V = \frac{d}{3}(A_1 + A_2 + \sqrt{A_1 A_2}) \quad ...(9.1)$$

Prismoidal formula, $$V = \frac{d}{6}(A_1 + A_2 + 4A_m) \quad ...(9.2)$$

STREAM FLOW ROUTING

In a stream channel (river) a flood wave may be reduced in magnitude and lengthened in travel time i.e., attenuated, by storage in the reach between two sections. The storage in the reach may be divided into two parts-prism storage and wedge storage, Fig. 9.5, since the water surface is not uniform during the floods. The volume that would be stored in the reach if the flow were uniform throughout, i.e., below a line parallel to the stream bed, is called 'prism storage' and the volume stored between this line and the actual water surface profile due to outflow being different from inflow into the reach is called 'wedge storage'. During rising stages the wedge storage volume is considerable before the outflow actually increases, while during falling stages inflow drops more rapidly than outflow, the wedge storage becoming negative.

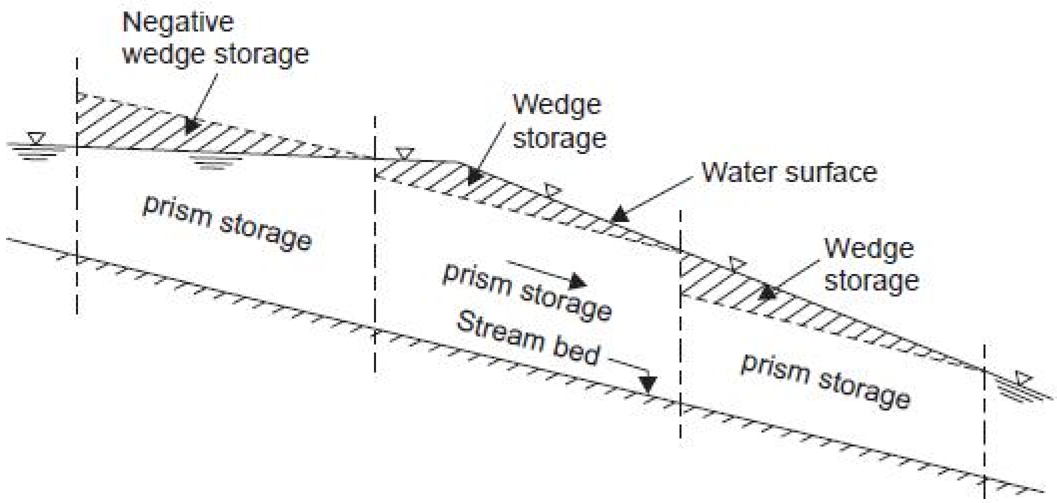


Fig. 9.5 Storage in a stream channel during a flood wave

In the case of stream-flow routing, the solution of the storage equation is more compli- cated, than in the case of reservoir routing, since the wedge storage is involved. While the storage in a reach depends on both the inflow and outflow, prism storage depends on the outflow alone and the wedge storage depends on the difference (I – O). A common method of stream flow routing is the Muskingum method (McCarthy, 1938) where the storage is ex- pressed as a function of both inflow and outflow in the reach as

$$S = K\,[xI + (1 - x)\,O] \qquad \text{...(9.9)}$$

where K and x are called the Muskingum coefficients (since the Eq. (9.9) was first developed by the U.S. Army Corps of Engineers in connection with the flood control schemes in the Muskingum River Basin, Ohio), K is a storage constant having the dimension of time and x is a dimensionless constant for the reach of the river. In natural riverchannels x ranges from 0.1 to 0.3. The Eq. (9.9) in most flood flows approaches a straight line. Trial values of x are as- sumed and plots of 'S vs. [xI + (1 – x) O]' are in the form of storage loops; for a particular value of x, the plot is a straight line and the slope of the line gives K. If S is in cumec-day and I, O are in cumec, K is in day.

FLOOD FREQUENCY METHODS

For the annual flood data of Lower Tapti River at Ukai (30 years: 1939–1968) in Example 8.5 the flood frequencies of 2–, 10–, 50–, 100–, 200–, and 1000-year floods have been worked out below by the probability methods developed by Fuller, Gumbel, Powell, Ven Te Chow, and stochastic methods, and the flood frequency curves are drawn on a semi-log paper as shown in Fig. 15.1. It can be seen that the Gumbel's method gives the prediction of floods of a particular frequency exceeding the observed floods by a safe margin and can be adopted in the design of the structure.

1. Fuller's formula. $Q_T = Q\,(1 + 0.8 \log T)$ *...(15.1) From Table 8.5,* $Q = 14.21$ *thousand cumec (tcm)*

T	*log T*	*0.8 log T*	$Q_T = Q\,(1 + 0.8 \log T)$ *(tcm)*
1000	3.0	2.4	48.3
200	2.3010	1.8408	40.4
100	2.0	1.6	37.0
50	1.6990	1.3592	33.5
10	1.0	0.8	25.6
2	0.3010	0.2408	17.35

2. Gumbel's method. *According to the extreme value distribution, the probability of occurrence of a flood peak Q, is given by*

$$P = 1 - e^{e^{-y}}$$

the reduced variate y is given by

3. Ven Te Chow method. *Another modification of the Gumbel's method was made by V.T. Chow by using the frequency factor. The equation is*

$$Q_T = a + bX_T \qquad \text{...(15.12)}$$

where

$$X_T = \log\left[\log \frac{T}{T-1}\right] \qquad \text{...(15.12 a)}$$

a, b = parameters estimated by the method of moments from the observed data. The following equations are derived from the method of least squares.

$$Q = an + b \quad X_T$$

$$(QX_T) = a \quad X_T + b \quad (X_T^2) \qquad \text{...(15.13)}$$

from which a and b can be solved.

In this method, a plotting position has been assigned for each value of Q when arranged in the descending order or magnitude of flood peaks. For example, if an annual flood peak Q_T *has a rank m, its plotting position*

STOCHASTIC METHOD

Anual Floods: *The methods described earlier were either probabilistic or deterministic, and did not consider the element of time which is possible only by the stochastic approach. Work in the field of stochastic hydrology has been introduced in USA by Yevdjevich, Ven Te Chow and others. One of the well known equations based on annual flood data using Poisson probability law and theory of sums of random number of random variables is*

$$Q_T = Q_{min} + 2.3\ (\bar{Q} - Q_{min})\ \log \left[\frac{n_f}{n} \cdot T\right] \qquad \ldots(15.17)$$

where $T = \dfrac{n}{m}$

n_f *= number of recorded floods, counting only one for the same flood peak occuring in different years.*

Computations are made by using Eq. (15.17) for the lower Tapti river at Ukai, below. Here $n_f = n = 30$, $\bar{Q} = 14.21$ *tcm,* $Q_{min} = 3.68$ *tcm*

$$Q_T = 3.68 + 2.3\ (14.21 - 3.68)\ \log \left[\frac{30}{30} \cdot T\right]$$

Drought management & water harvesting

Introduction

Drought is a deficiency in precipitation over an extended period, usually a season or more, resulting in a water shortage causing adverse impacts on vegetation, animals, and/or people. It is a normal, recurrent feature of climate that occurs in virtually all climate zones, from very wet to very dry. Drought is a temporary aberration from normal climatic conditions, thus it can vary significantly from one region to another. Drought is different than aridity, which is a permanent feature of climate in regions where low precipitation is the norm, as in a desert.

Human factors, such as water demand and water management, can exacerbate the impact that drought has on a region. Because of the interplay between a natural drought event and various human factors, drought means different things to different people. In practice, drought is defined in a number of ways that reflect various perspectives and interests. Below are three commonly used definitions:

Defining a drought is difficult because of the word normal. In many areas, normal conditions generally mean conditions that do not deviate from long-term averages. However, these averages themselves can change over time.

Types of Drought

Meteorological Drought

Meteorological drought is usually defined based on the degree of dryness (in comparison to some "normal" or average) and the duration of the dry period. Drought onset generally occurs with a meteorological drought. A lack of precipitation is the most common definition of drought and is usually the type of drought referred to in news reports and the media. Most locations around the world have their own meteorological definition of drought based on the climate normals in the area. A normally rainy area that gets less rain than usual can be considered in a drought.

Agricultural Drought

Agricultural drought links various characteris- tics of meteorological (or hydrological) drought to agricultural impacts, focusing on precipita- tion shortages, soil water deficits, reduced ground water or reservoir levels needed for irrigation, and so forth. When soil moisture becomes a problem, the agricultural industry is in trouble with drought. Shortages in precipitation, changes in evapo-transpiration, and reduced ground water levels can create stress and problems for crops.

Hydrological Drought

Hydrological drought usually occurs following periods of extended precipitation shortfalls that impact water supply (i.e., streamflow, reservoir and lake levels, ground water), potentially re- sulting in significant societal impacts. Because regions are interconnected by hydrologic systems, the impact of meteorological drought may extend well beyond the borders of the precipitation-deficient area. Many watersheds experience depleted amounts of available water. Lack of water in river systems and reservoirs can impact hydroelectric power companies, farmers, wildlife, and communities. Hydrological drought usually occurs following periods of extended precipitation shortfalls that impact water supply (i.e., streamflow, reservoir and lake levels, ground water), potentially re- sulting in significant societal impacts. Because regions are interconnected by hydrologic systems, the impact of meteorological drought may extend well beyond the borders of the precipitation-deficient area.

Causes Droughts

The cause of droughts is easily understood, but hard to prevent. Depending on the location, crop failures, famine, high food prices, and deaths can occur. One of the scariest parts of a drought is the onset time. Unlike other forms of severe weather or natural disasters, droughts often develop slowly

Droughts are caused by a depletion of precipitation over time. Unlike a dry spell, prolonged lack of rain will cause regions around the world to slowly dry out. Because of the slow onset of droughts, their cost is often only estimated. Frequently, droughts are billion dollar weather events and are one of the top three threats to population in the world (along with famine and flooding).

Sometimes a drought takes decades to develop fully and predicting droughts is difficult. The frequency of droughts in the United States is literally every year. In other words, somewhere in the US in any given year, a drought is occurring. Droughts are completely natural, but their devastation can be far-reaching and severe. Atmospheric conditions such as climate change, ocean temperatures, changes in the jet stream, and changes in the local landscape are all culprits in the long story of the causes of droughts.

We already know that a drought occurs when not enough rain falls to the ground. However, water vapor condenses only if air rises into the colder regions of the atmosphere. If the air doesn't rise, then no rain will form. When there is high air pressure, air falls instead of rising. With the air pressing down in a high pressure zone, no currents of water vapor are carried upward. As a result, no condensation occurs, and little rain falls to earth. In addition, high-pressure areas push clouds and air currents downward and away, resulting in sunny, cloudless weather. Low-pressure systems see more cloudy, stormy weather.

Usually, however, we experience both high- and low-pressure systems. It is normal for a high-pressure system to pass over an area and move on, being replaced by a low-pressure system. However, when a high-pressure system is stalled, the sunny weather can drag on for days. If it keeps on going, the result is a drought.

High-pressure systems can be stalled by jet streams, wide bands of fast-moving air (up to 335 miles per hour) in the upper atmosphere. Masses of air that usually move from place to place can be locked in one area by jet streams.

Unusual currents of cold and warm water in the ocean can also stall a high-pressure system. In the Pacific, a warm water current known as El Nino brings low-pressure systems that cause hurricanes and other violent storms to North America, while a cold water current known as La Nina brings drought. In Asia, the opposite occurs, with El Nino bringing drought and La Nina stormy weather.

Or droughts occur because water vapor is not brought by air currents to the right areas at the right times. Water that evaporates from the oceans is brought inland by wind to regions where it is needed. However, sometimes those winds are not strong enough. In the eastern United States, moisture is carried up from the Gulf of Mexico by northward blowing winds. This moisture is then pushed by other winds until it reaches the Midwest. This water then falls to the ground, supporting the farms in that region. However, if the winds don't blow at the right time, in the right direction, or with enough force, the moisture falls in other areas and that Midwest region suffers from drought. A similar phenomenon occurs in southeast Asia. Usually, summer winds known as monsoons carry water vapor north from the

Indian Ocean inland, providing desperately needed rain. Sometimes, however, instead of blowing from north to south, they blow east to west. When that happens, the vapor doesn't leave the Indian Ocean and many people suffer from the resulting droughts.

Mountains can prevent wind from blowing moisture to needed regions. As air is moving past a mountain range, it is forced to rise in order to pass over the peaks. However, as the air rises, it becomes colder and the vapor condenses into rain or snow. The rain then falls on that side of the mountain, known as the windward side (the side that is turned toward the wind). When the air mass finally makes it over the mountain, it has lost much of its vapor. This is another reason why many deserts are found on the side of a mountain facing away from the ocean. This phenomenon is known as the rain shadow effect.

Impacts of Drought

While droughts do not often cause deaths in the United States, the Dust Bowl in the US Midwest is one example of the devastation that can occur. This site has a great list of otherfamous droughts.

1. *There are three main ways droughts impact lives and communities. First, the economic impacts of drought include losses in the timber, agricultural, and fisheries communities. Many of these losses are then passed on to consumers in the form of higher commodity pricing.*
2. *Next social impacts include increased chance of conflict over commodities, fertile land, and water resources. Other social impacts include abandonment of cultural traditions, loss of homelands, changes in lifestyle, and increased chance of health risks due to poverty and hygiene issues.*
3. *Finally, the environmental impacts of drought include loss in species biodiversity, migration changes, reduced air quality, and increased soil erosion.*

Other parts of the world experience long periods without rains as well. Even during monsoon season, areas that depend on the seasonal rains will often experience drought if the monsoon rains fail. Once crops fail, famine can become a major problem. In some African countries, rain rituals are often used to try and thwart the dry seasons and bring on the rain. While it is no cure, modern technology has developed ways to help see potential famine situations as satellites see famine conditions from space.

Why Conserve Water?

Water conservation is an ongoing component of water resource management.

- *The Washington State Department of Health's Municipal Water Conservation Analysis and Recommendations states:*

Ensuring the efficient use of our limited water resources is a key component to the overall management of the state water resources and to salmon recovery efforts. Efficient water use benefits state natural resources by keeping as much water as possible in the natural environment. It also benefits water utilities and local governments by lowering water demands that may require costly new source development projects and by helping to ensure that water is available to meet economic and population growth consistent with local Growth Management Act planning efforts.

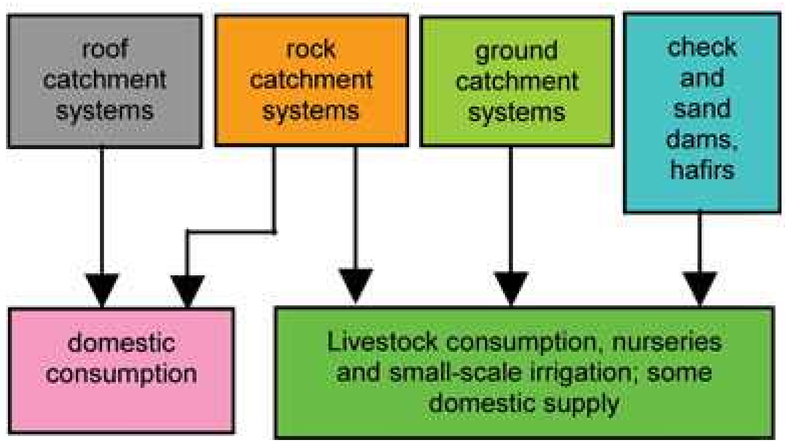
What is rainwater harvesting?

Rainwater harvesting is a technology used to collect, convey and store rain for later use from relatively clean surfaces such as a roof, land surface or rock catchment. The water is generally stored in a rainwater tank or directed to recharge groundwater. Rainwater infiltration is another aspect of rainwater harvesting playing an important role in stormwater management and in the replenishment of the groundwater levels. Rainwater harvesting has been pr

acticed for over 4,000 years throughout the world, acticed for over 4,000 years throughout the world, traditionally in arid and semi-arid areas, and has provided drinking water, domestic water and water for livestock and small irrigation. Today, rainwater harvesting has gained much on significance as a modern, water-saving and simple technology.

The practice of collecting rainwater from rainfall events can be classified into two broad categories: land-based and roof-based. Land-based rainwater harvesting occurs when runoff from land surfaces is collected in furrow dikes, ponds, tanks and reservoirs. Roof-based rainwater harvesting refers to collecting rainwater runoff from roof surfaces which usually provides a much cleaner source of water that can be also used for drinking.

Gould and Nissen-Petersen (1999) categorised rainwater harvesting according to the type of catchment surface used and the scale of activity (Figure 1).



Why rainwater harvesting?

In many regions of the world, clean drinking water is not always available and this is only possible with tremendous investment costs and expenditure. Rainwater is a free source and relatively clean and with proper treatment it can be even used as a potable water source. Rainwater harvesting saves high-quality drinking water sources and relieves the pressure on sewers and the environment by mitigating floods, soil erosions and replenishing groundwater levels. In addition, rainwater harvesting reduces the potable water consumption and consequently, the volume of generated wastewater.

Application areas

Rainwater harvesting systems can be installed in both new and existing buildings and harvested rainwater used for different applications that do not require drinking water quality such as toilet flushing, garden watering, irrigation, cleaning and laundry washing. Harvested rainwater is also used in many parts of the world as a drinking water source. As rainwater is very soft there is also less

consumption of washing and cleaning powder. With rainwater harvesting, the savings in potable water could amount up to 50% of the total household consumption.

Components of a rooftop rainwater harvesting system

Although rainwater can be harvested from many surfaces, rooftop harvesting systems are most commonly used as the quality of harvested rainwater is usually clean following proper installation and maintenance. The effective roof area and the material used in constructing the roof largely influence the efficiency of collection and the water quality.

Rainwater harvesting systems generally consist of four basic elements:

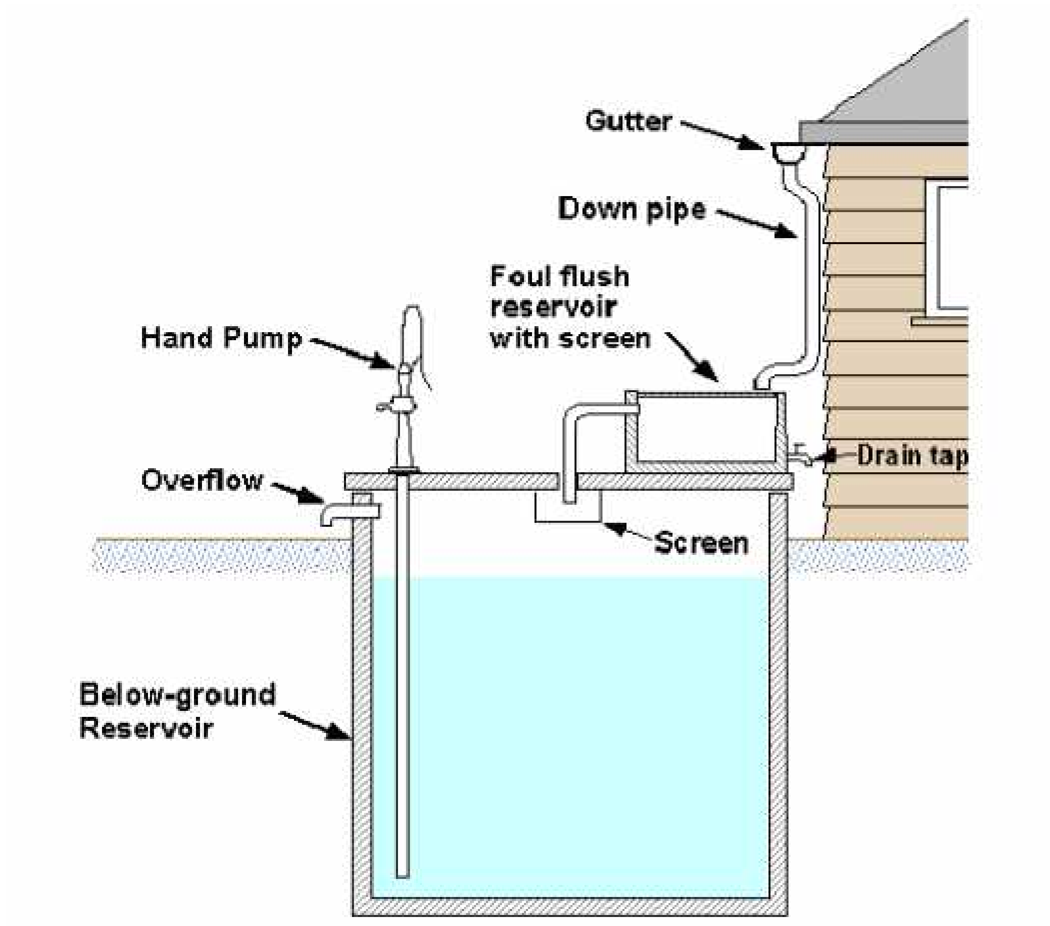
(1) a collection (catchment) area

(2) a conveyance system consisting of pipes and gutters

(3) a storage facility, and

(4) a delivery system consisting of a tap or pump.

Figure 2 shows a simple schematic diagram of a rooftop rainwater harvesting system including conveyance and storage facilities.



*1) **A collection or catchment** system is generally a simple structure such as roofs and/or gutters that direct rainwater into the storage facility. Roofs are ideal as catchment areas as they easily collect large volumes of rainwater.*

The amount and quality of rainwater collected from a catchment area depends upon the rain intensity, roof surface area, type of roofing material and the surrounding environment. Roofs should be

constructed of chemically inert materials such as wood, plastic, aluminium, or fibreglass. Roofing materials that are well suited include slates, clay tiles and concrete tiles. Galvanised corrugated iron and thatched roofs made from palm leaves are also suitable. Generally, unpainted and uncoated surface areas are most suitable. If paint is used, it should be non-toxic (no lead-based paints).

(2) ***A conveyance system*** *is required to transfer the rainwater from the roof catchment area to the storage system by connecting roof drains (drain pipes) and piping from the roof top to one or more downspouts that transport the rainwater through a filter system to the storage tanks. Materials suitable for the pipework include polyethylene (PE), polypropylene (PP) or stainless steel.*

Before water is stored in a storage tank or cistern, and prior to use, it should be filtered to remove particles and debris. The choice of the filtering system depends on the construction conditions. Low-maintenance filters with a good filter output and high water flow should be preferred. "First flush" systems which filter out the first rain and diverts it away from the storage tank should be also installed. This will remove the contaminants in rainwater which are highest in the first rain shower.

(3) ***Storage tank or cistern*** *to store harvested rainwater for use when needed. Depending on the space available these tanks can be constructed above grade, partly underground, or below grade. They may be constructed as part of the building, or may be built as a separate unit located some distance away from the building.*

The storage tank should be also constructed of an inert material such as reinforced concrete, ferrocement (reinforced steel and concrete), fibreglass, polyethylene, or stainless steel, or they could be made of wood, metal, or earth. The choice of material depends on local availability and affordability. Various types can be used including cylindrical ferrocement tanks, mortar jars (large jar shaped vessels constructed from wire reinforced mortar) and single and battery (interconnected) tanks. Polyethylene tanks are the most common and easiest to clean and connect to the piping system. Storage tanks must be opaque to inhibit algal growth and should be located near to the supply and demand points to reduce the distance water is conveyed.

Water flow into the storage tank or cistern is also decisive for the quality of the cistern water. Calm rainwater inlet will prevent the stirring up of the sediment. Upon leaving the cistern, the stored water is extracted from the cleanest part of the tank, just below the surface of the water, using a floating extraction filter. A sloping overflow trap is necessary to drain away any floating matter and to protect from sewer gases. Storage tanks should be also kept closed to prevent the entry of insects and other animals.

(4) ***Delivery system*** *which delivers rainwater and it usually includes a small pump, a pressure tank and a tap, if delivery by means of simple gravity on site is not feasible.*

Disinfection of the harvested rainwater, which includes filtration and/or ozone or UV disinfection, is necessary if rainwater is to be used as a potable water source.